INDIAN STORAGE RESERVOIRS WITH EARTHEN DAMS:

BEING

A PRACTICAL TREATISE ON THEIR DESIGN AND CONSTRUCTION,

BY

WILLIAM LUMISDEN STRANGE, M.Inst.C.E.,

CHIEF ENGINEER IN SIND (RETIRED)

Author of *Irrigation, Roads and Buildings*, and *Indian Engineering*

THIRD AND ENLARGED EDITION

17 *PLATES AND* 63 *ILLUSTRATIONS*

"For in the wilderness shall waters break out, and streams in the desert. And the parched ground shall become a pool, and the thirsty land springs of water."—ISAIAH xxxv, 6, 7.

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PREFACE.

It is probable that one of the first results of the inquiries made by the Indian Irrigation Commission will be the construction of storage reservoirs to serve parts of the country which are peculiarly liable to drought. Indeed, several such schemes have already been submitted, and investigations in respect to others —some of the largest magnitude—are now in progress. Seeing that but few works of this class have been constructed anywhere in recent years, and that they are now being commenced for the first time in certain provinces, it occurred to me that a book treating of the different problems which present themselves, both in the design and in the construction of storage reservoirs, would be of use as a guide to those entrusted with their execution. I have written the following pages, however, in the hope that they may prove helpful both to those who have already had to deal with this class of work as well as to those who have not yet had any connection with it.

My own experience has been gained in the Bombay Presidency, and I have endeavoured to describe the practice which usually obtains there, as well as certain modifications and improvements of it which appear to me to be desirable. Although that experience and practice are necessarily of a local character, it is not unlikely that similar conditions to those which prevail in Bombay will be found in other parts of the Empire which lend themselves to the construction of storage

reservoirs, and I therefore trust that this work will be of general utility in India.

Mr. Fanning observes[1]: "An earthwork embankment appears to the uninitiated the most simple of all engineering constructions, the one feature that demands least of educated judgment and experience." I hope that the following pages will show that this opinion of the uninitiated is entirely erroneous, and that for the proper design and construction of such a work a very considerable amount of skill and attention is absolutely necessary if success is to be attained.

The fact is that in the case of earthen embankments we are dealing with a material which is unsatisfactory and unreliable unless carefully treated, whereas, if it is properly utilised it has peculiar advantages of its own and is permanent to an eminent degree, as some of the oldest works in the world testify. Such embankments are, moreover, the cheapest structures whereby water can be stored, and they are particularly suitable for the employment of the large amount of unskilled labour which is available in ordinary times in India, and for which work has to be found in times of scarcity. I have therefore described in detail the precautions which it is most desirable to take—precautions which involve careful supervision rather than greatly increased expenditure. Naturally, they are needed more in the case of large and important works than in that of small and relatively insignificant ones.

Reservoirs in India are required either for irrigation or for the water-supply of towns. The former class, being the more important from an engineering point of view, has been described in detail, while the latter has

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[1] "A Treatise on Hydraulic and Water-Supply Engineering." 15th edition, 1902. Page 334

been subsequently dealt with at less length. The different parts of the complete project—the dam, the waste-weir, and the outlet—form the subjects of separate chapters, which, for the sake of easy reference, have been divided into numbered sections and paragraphs. The general conditions which have to be taken into account are discussed in the initial chapter, while the final one is devoted to miscellaneous subjects; at the end has been added a series of appendices which deal with and illustrate the general matters noticed in the earlier part.

The book is intended to be entirely practical, and, I believe, covers ground which has not been occupied before, as descriptive engineering works dealing with a variety of subjects do not usually go into complete detail in respect to any one of them. It has been my object to treat this one class of work as exhaustively as possible, so that an engineer not previously acquainted with it should, after studying these pages, be able to design correctly, and to construct securely, a storage reservoir with an earthen dam.

I am indebted to the Institution of Civil Engineers for permission to make free use of Paper No. 2996, which I contributed to Volume CXXXII. of their "Minutes of Proceedings," and of another Paper, which was not published, both of which deal with the same subject. I am also much obliged to Sir Thomas Higham, K.C.I.E., and to Mr. R. B. Buckley, C.S.I., for permission to make use of their books. I have acknowledged in the text my obligations to the other authorities whom I have consulted.

I have endeavoured to distinguish clearly between matters of ordinary practice accepted in Bombay and my own suggestions and recommendations, but, as the

book is written impersonally, it may be as well, in order to avoid any chance of misapprehension, to say that the following are put forward from the results of my own experience :—

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The recommendation for the adoption of the "Headwall on the centre line of the dam" to secure the subsidiary uses of the outlet	206
The sanitary treatment of catchment areas [1]	227

The appendices, with the exception of Nos. 1, 6, 9, 23, 24, and 25, are original.

As it is generally found easier to follow a concrete example than an abstract description, I have, in order to make the latter clearer, taken for the former the project for the Máládevi Tank, which I drew up some years ago. This large scheme comprises practically all the features

[1] Also, safety flood cuts, paragraph 163, and the double control over outlet sluices, paragraph 215.

with which it is necessary to deal in the design and construction of a reservoir with a high earthen dam.

In conclusion, I may state for the benefit of engineers without Indian experience that the following are the conditions which have to be met in India. The rainfall is almost entirely confined to the monsoon months, and is very capricious in amount and intensity. The rest of the year is generally characterised by a total absence of rain, and during this period a fall seldom occurs sufficient to produce replenishment. Constructional work is practically impossible during the rains, and the programme for the execution of the work has therefore to be confined to the seven fair-weather months of the year. Owing to these climatic conditions, and also to the tropical heat and storms, a large amount of storage has to be effected, and a large provision for the safe discharge of floods has to be made.

The labour available is unskilled, and the amount of skilled supervision is limited.

W. L. STRANGE.

SIMLA, *April*, 1903.

I HAVE to acknowledge the great assistance that I have received from Mr. G. W. Herdman, B.Sc., in the revision of the proofs, and from Mr. Buckley in arranging for the publication of the book.

W. L. S.

PRETORIA, *February* 1904.

PREFACE TO SECOND EDITION.

FOR this edition the original one has been thoroughly revised and amplified where necessary. A few new paragraphs, appendices, plates and figures have been added; these have been numbered so that their numbering does not interfere with that of the original ones. The figures in the text have been redrawn and an index made. The Máládevi project has now been abandoned, *vide* the footnote to paragraph 43A.

W. L. S.

READING, *April*, 1913.

PREFACE TO THIRD EDITION.

ALTHOUGH this book has more than attained its majority, its continued sale indicates it is still of use to its small public, and therefore this third edition is issued in the hope that it will prove of service. For this edition the text has again been revised and slightly expanded as required. A few new paragraphs, etc., have been added: these are distinguished by the letter "A" for those first included in the second edition, and by "B" for those now inserted in the third edition, so that the numbering of those of the original book is preserved.

W. L. S.

WORTHING, *July*, 1927.

Note.—All costs quoted are at pre-war rates.

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GLOSSARY OF ABBREVIATIONS AND INDIAN WORDS.

F. S.	= full-supply.
F. S. L.	= full-supply level.
H. F. L.	= high-flood level.
M. S. L.	= mean sea level.
R. L.	= reduced level.
T. P.	= trial pit.
Cusecs	= cubic feet per second.
Cft.	= cubic feet.
Mill. cft.	= million cubic feet.
Rft.	= running feet.
Sft.	= square feet.

Anna	= $\frac{1}{16}$th rupee = 1*d*.
Babhul	= a common hard and tough wood (Acacia Arabica).
Bhistie	= a water-carrier.
Coolie	= a works labourer.
Geru	= a soft clay rock.
Ghát	= main mountain range.
Kankar	= nodular hydraulic limestone.
Karal	= a grey marly soil charged with salts.
Kárkun	= a works clerk or timekeeper.
Kharif	= the season from February 15th to October 14th.
Maistry	= a works foreman.
Mán	= a hard brown clay soil.
Monsoon	= the rainy season.
Muccadum	= a works gang-man.
Muram	= disintegrated trap rock.
Nulla	= a water-course.
One lakh	= 100,000.
Peon	= an office messenger.
Pie	= $\frac{1}{12}$th anna = $\frac{1}{12}$th penny.
Rabi	= the season from October 15th to February 14th.
Rupee	= 1*s*. 4*d*.
Shádu	= a white marly soil
Súp	= a basket scoop for bailing water.
Tamarisk	= a jointed evergreen reedy plant growing on riverain lands (Tamarix Gallica).
Tank	= a storage reservoir.

ERRATA.

ge 238, line 2: *For* $D = a\sqrt{r}\,.\,c_2 z\,\sqrt{s}$ *read* $D = a\sqrt{r}\,.\,c_2\,\sqrt{s}$.

ge 268, line 13: *For* Khavregat *read* Kharegat.

ge 388, Section V, item 34:

r "34. The Berm	110	87
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INDIAN STORAGE RESERVOIRS WITH EARTHEN DAMS.

CHAPTER I.

THE RESERVOIR.

1[B]. Introductory.—Reservoirs are works for the storage of water which, usually, is impounded for subsequent utilisation. If that water is due to storm flow which would otherwise run to waste, its storage will be a new asset to the country and should be permissible free of liability to others: if, however, its storage interferes with vested interests, compensation for it should be made either by money or water.

All reservoirs naturally store water, but the name "storage reservoirs" is given in India to works for irrigation supply, and is adopted for them in this book. There are other varieties of reservoirs, such as those for the water supply of towns, general utility (power and light), manufacturing purposes, compensation, and for flood regulation: as works they have their principal meteorological, hydraulic and constructional features in common—they differ from each other in the way in which their storage is utilised. Reservoirs are formed by dams and the usual types of these may be classed as earthen, and masonry or composite (consisting of separate sections of the other two): there are other types more modern and probably less permanent. This book is limited in detail to the type first mentioned.

When the construction of a storage reservoir is in contemplation, the following are among the principal matters which should be considered :—

I. Construction.

The geological conditions affecting the construction of the works ;

The suitability of the natural features for the dam, waste-weir and outlet ;

The selection of the type of dam—earthen, masonry or composite ;

The availability and cost of labour and materials ;

The conditions of the working season.

II. Storage.

The sufficiency of the catchment and the facilities for its extension ;

The nature, sufficiency and variation of the rainfall ;

The character of the catchment in respect to yield, high-flood run-off, silting, etc. ;

The features of the storage basin, whether open or confined, pervious or impervious, etc.

III. Irrigation.

The necessity, or not, for irrigation ;

The suitability of the climate, soil and water ;

The existence, or not, of skilled and enterprising irrigators ;

The situation of the irrigable land with respect to the storage ;

The facilities for the construction of the canal ;

The extent and the degree of compactness of the irrigable area ;

The "duty" of water probable ;

The availability, or not, of manure.

IV. Financial.

The probable cost of the works;
The probable amount of the rate of storage;
The financial status of the future irrigators;
The irrigation rates assessable;
The financial prospects of the scheme;
The markets and communications available or prospective;
The further developments possible.

I. Selection of the Site.[1]

1. Selection of the Site of the Dam.—When choosing a site for a dam it should be remembered that generally the longitudinal section indicates the cost of the scheme; the sites available for waste-weirs and outlets, its feasibility; and the nature of the reservoir basin, whether open or restricted, the relative cost of storage.

The best site for a dam is usually one which has ridges running down from high land on both sides to the stream to be impounded; such ridges will greatly reduce the cost, as the sectional area of a dam varies roughly as $2\frac{1}{2}$ times the square of its height (para. 73, p. 107); long, low dams are thus frequently cheaper than short, high ones, and are also much safer.

A ridge, moreover, offers facilities for the proper drainage of the dam. The ridge should, however, not be very narrow, for, if it is, it may be liable to leak; may not allow space for the future raising of the embankment, should that ever become necessary; and may also lead to greater cost and difficulty in construction.

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[1] The numbering of paragraphs 1–5 in the first edition has been altered in this one.

It should be seen that the longitudinal section provides proper sites for the outlet, temporary (para. 134, p. 185) and permanent waste-weirs, and for "breaching sections" of the dam (para. 75, p. 109), at which, in the event of the work being breached, the resulting flood will do the least amount of damage. Generally, the best site is one at which the gorge part of the dam is separated from the flank embankment by a hillock rising above the level of the top of the dam, as then these portions of the work can be completed independently of each other. This formation is peculiarly suited for composite dams (para. 54, p. 77), as the gorge portion can be constructed in masonry, while the flank can be made in embankment.

When the basin of the reservoir is underlain by permeable strata, or when the site of the dam consists of permeable or insecurely-bedded strata, careful geological investigation thereof is necessary, so that, if possible, proper precautions may be taken to remedy these defects, or so that the site may be rejected if the natural conditions are such as will make it dangerous or impracticable to construct the works thereon. Thereafter engineering examination is required to prove the sufficiency of the foundations.

The abundance and proximity of all materials required for construction—such as the proper class of soil for the embankment, water, sand, lime, wood for fuel, and stone for masonry and pitching—should also be taken into account. It is generally not practicable economically to construct an ordinary earthen dam if sufficient good soil does not exist within half a mile of its site.

2. Selection of the Storage Basin.—When a site has been found which satisfies purely replenishment and

penetrates more deeply and produces springs (which supply the fair-weather flow of streams), but in India these generally give an insignificant amount of discharge. The chief loss in respect to stream flow is due to direct or indirect evaporation. Finally, another part runs off the surface to the drainage lines; it is this which in India produces the monsoon flow of streams, and constitutes the large bulk of the supply available for replenishing reservoirs. The proportions in which the total rainfall will thus be disposed of will depend (as described in para. 7, p. 10), upon the nature of the surface, the intensity and amount of the showers and the intervals at which they occur. Rainfall, including snowfall and dew, is the sole source of supply for irrigation.

13. Variations in Rainfall and determination of the Run-off therefrom.—In England, where the total annual rainfall does not vary greatly, the following laws have been put forward. "Glaisher's law" is that the average fall of three consecutive years yielding the least fall may be taken as the average minimum fall. "Hawksley's law" is that from the average fall of twenty years one-sixth should be deducted to obtain the average of the three years of minimum fall. These two laws should agree in any selected case.

In India such general rules have not been devised, and, if devised, would probably not hold good owing to the extreme variations that there exist—on the one hand between famine years, when there may be a very small fall of rain, and, on the other, years of excessive fall, when the precipitation may greatly exceed that during an average year. Rainfall varies from year to year at the same place, and also from place to place during the same year. Despite this

variation, meteorologists believe that the average annual fall during cycles of about 35 years does not vary by more than 2 per cent.

In years of deficient fall, to mitigate the effects of which storage reservoirs are chiefly required, not only is the total precipitation small, but the proportion of it which runs off, compared to that of average years, may considerably diminish (*vide* Appx. 2, p. 349). Thus the total average yield should not be calculated directly from the total average annual rainfall. The behaviour of each catchment in this respect requires individual study, as each has its local peculiarities, but much may be learnt from that of similarly situated catchments generally resembling the one under consideration. It is therefore highly important in connection with proposed works, to maintain observations relating to this subject on all existing works, and for as long as possible.

For existing works it is easy to measure the total run-off by the replenishment received until the time when the reservoir fills, and this is done in the Bombay Presidency. It is, however, equally, if not more, important to continue the observations throughout the whole year, but this has not usually been done there. In order to determine the run-off during the whole year it will be necessary to ascertain the total discharge passed off by the waste-weir, either by frequent observations of the depth flowing over it, or by obtaining a continuous diagram of these depths by means of a self-recording instrument, such as the "automatic water-level recorder," made by the Glenfield Co., Kilmarnock. From such a diagram another diagram of discharges can be prepared by plotting the rate of discharge corresponding to each depth on the

weir crest and the duration of flow, and the total discharge can be ascertained by measuring the area included by this second diagram. Or, a recorder designed to register the amount of the discharge automatically (also made by the same firm) may be used.

For proposed works the total run-off of the year is ascertained by gauging the stream once or twice daily during ordinary flow, and at more frequent intervals during floods, when the level changes rapidly (para. 46, p. 68). It is, however, difficult to arrange for such observations at night and during floods of great intensity, and it is therefore advisable to instal automatic discharge recorders in order to obtain more accurate results.

14. Necessity for maintaining several Rain-gauges.—To compare the average rainfall even on a small catchment with the run-off it is not sufficient to maintain a single rain-gauge, unless the precipitation is uniform over it. At the Khás tank in the Satára district, Bombay, the rain-gauge, which is at the site of the dam, records only the minimum precipitation; the maximum fall occurs on that part of the watershed which is situated on the crest of the gháts. The variation between the two is very great, although the catchment is very small (1·97 square miles). The result is that the total annual run-off sometimes exceeds the amount which would be produced were the rainfall as gauged (near the dam) uniform over the catchment and wholly to flow off it. Generally speaking, in hilly tracts the rainfall at the dam site is always considerably less than the average fall on the catchment. It is therefore necessary in order to ascertain this average fall, to have numerous rain-gauges properly distributed over

the catchment: the number of gauges required depends upon the physical characteristics of the drainage area (para. 11, p. 16). The average fall is moreover not the arithmetical mean of the registration of the different gauges: to determine it correctly, the fall registered by each instrument should be multiplied by the area it represents and the sum thus arrived at should be divided by the area of the whole catchment.

15. Examples of the Proportion of Run-off to Rainfall.—A few instances are given below of the percentage of run-off to rainfall.

Observations[1] recorded at Nágpur and at Jubbulpore, in the Central Provinces, showed that in ordinary years the total run-off from a rocky catchment of 5 or 6 square miles was 40 per cent. of the total rainfall. At the Oopahalli tank, in Mysore, the total run-off from 1878 to 1883 varied from 10 to 19 per cent. only of the total rainfall; at the Hulsar tank, at Bangalore, during three years the percentage was 14.

In the Bombay Administration Report of the Public Works Department for 1890–91 is a statement showing the behaviour of the catchments of 19 tanks in the Deccán during the monsoon rainfall, which varied generally from 20 to 30 inches, but amounted in some cases to as little as 4 or 5 inches. The 114 observations showed that there were :—

26	cases where	the flow-off	was	less than 10	per cent. of the total rainfall.
44	,,	,,	,,	between 10 & 20	
25	,,	,,	,,	,, 20 & 30	
19	,,	,,	,,	above 30	

[1] Buckley's "The Irrigation Works of India," 2nd edn., pp 60 and 61.

In Bombay it is generally assumed that the run-off will be only 25 per cent. of the monsoon rainfall, when this varies from 20 to 25 inches.

The record of observations below is taken from the Bombay Irrigation Revenue Reports.[1] These tanks are situated in the "Famine Zone." It will be noticed how small is the run-off percentage; that in a year of abnormally small rainfall it practically vanishes; that the run-off does not vary directly with the total monsoon fall, as it is due principally to individual heavy storms; and that, (owing to their physical peculiarities), the average percentage run-off differs considerably for the three catchments although the average rainfall on them does not vary greatly.

Year	Mhasvad Tank Catchment Area, 484 sq miles		Ashti Tank Catchment Area, 92 sq miles		Ekruk Tank Catchment Area, 159 sq. miles	
	Monsoon Rainfall Inches	Percentage of Run-off	Monsoon Rainfall Inches	Percentage of Run-off	Monsoon Rainfall Inches	Percentage of Run-off.
1884–85	13·40	9 5	Abnormally small	1·9	20·39	17·8
1885–86	20·01	28 0	28·91	24·5	27·93	21·5
1886–87	28·88	15·5	29·21	16·5	33·05	22·6
1887–88	27 76	12 1	18·16	18·3	35·88	33·8
1888–89	21·00	10·5	13·89	11·7	24·75	19·8
Average	22·21	14·34	22·54	18·69	28·40	24·04

The following record is taken from the report of the Rushikulya project in Madras[2]:—

[1] Buckley's "The Irrigation Works of India," 2nd edn., p 63.
[2] *Ibid*, p. 63.

Year.	Mahánadi. Catchment, 900 sq miles		Rushikulya. Catchment, 850 sq miles	
	Rainfall of the Year. Inches.	Percentage of Run-off.	Rainfall of the Year. Inches	Percentage of Run-off
1868 . . .	44·1	5·0	44·1	6 3
1869 . . .	59·6	36·1	59·6	31·0
1870 . . .	55·5	51·3	52·7	74·2
1871 . . .	43·7	18·0	42·0	8·4
Average . .	50·73	29·60	49·6	32·2

The above shows that the percentage of run-off generally decreases with the total annual rainfall, but that it does not always do so. This point is noticed in paragraph 17.

The following table shows the effect of the state of saturation of the ground on the percentage of run-off. The observations were made on a catchment of about 100 square miles near Calcutta [1] :—

Month	Percentage of Run-off to Rainfall during the Month.		Remarks.
	In Years of Ordinary Rainfall.	In Years of Heavy Rainfall.	
June . .	5·0	10·0	At end of hot weather.
July . .	10·0	20·0	
August. .	25·0	50·0	Monsoon well established.
September .	40·0	50·0	
October .	40·0	50·0	End of Monsoon.

16. Tabular Statement of Rainfall and Run-off.—Careful experiments were made during the monsoon of

[1] Buckley's "The Irrigation Works of India," 2nd edn , p 63.

1872 by Mr. (now Sir Alexander) Binnie on the Ambájhari reservoir catchment near Nágpur, in the Central Provinces, and a table of the percentages of run-off to rainfall was published [1] by him. In Appendix 2, p. 349, is a similar table in which the percentages are given for a regularly ascending scale of rainfall, and under three conditions, viz., of a bad, of an average, and of a good catchment. The results are plotted in the diagram, Plate I.

This classification of catchments relates to their capabilities of producing runs-off, and the figures for a good catchment approximate to those given for Ambájhari. The table is based on general ideas, and it is hoped will prove useful. It would be of great utility were similar tables constructed by engineers for different actual catchments. The general result to be noticed from this table is that in years of deficient rainfall the percentage of run-off rapidly diminishes, so that, on account of the smallness both of the fall and of the percentage of run-off, the replenishment of a reservoir in such years is very little when compared with that in a year of good rainfall, or even in one of average fall.

17. The Effect of the Intensity of the Rainfall and the Saturation of the Ground on the Run-off.—The chief defect of such a table is that it is based on the annual monsoon fall, and not on the intensity and amount of the individual showers of rain which actually produced the run-off, and the condition of the catchment when those showers fell. Thus, con-considering the same catchment, two years with identically the same total rainfall may produce replenishments of greatly differing amount. It is

[1] "Minutes of Proceedings, Inst. C E," Vol. xxxix

evident that if the total rainfall is fairly distributed over sixty days it will produce much less run-off than if it fell during thirty days, and still less than if it occurred during only fifteen days. The proper way, therefore, is to take into account the intensity, or rate of precipitation, and the amount of the individual falls of reservoir-filling rain. It would be best to consider the hourly intensity, but, as this is seldom recorded, the daily intensity may, in default, be taken. A great advantage of automatic over ordinary rain-gauges is that they record the rate, as well as the amount, of the rainfall.

The intensity of the rainfall is not, however, the sole factor which determines the rate of run-off. Cases have been known where even heavy falls of rain at the beginning of the monsoon, or after long breaks in it, when the surface was dry and absorbent, have not produced any run-off whatever, while falls of the same or of less amount, when the soil was saturated, have caused floods. It is evident, therefore, that the degree of saturation of the surface must also be taken into account when estimating the proportion which the run-off will bear to the rainfall. For this reason, as a rule, post monsoon falls of ordinary intensity should not be taken into account (Appx. 2, col. 1, p. 349).

18. **Tabular Statement of Daily Runs-off.**—As far as is known, careful observations of this latter effect have not been recorded. The percentages of run-off will of course vary with the other conditions of the catchment, but the table on p. 27 is given as a rough approximation of what may be expected from an ordinary drainage area. The percentages will increase with a good, and diminish with a bad, one.

FIG. A

CURVES SHOWING EFFECT OF STATE OF GROUND ON RUN-OFF

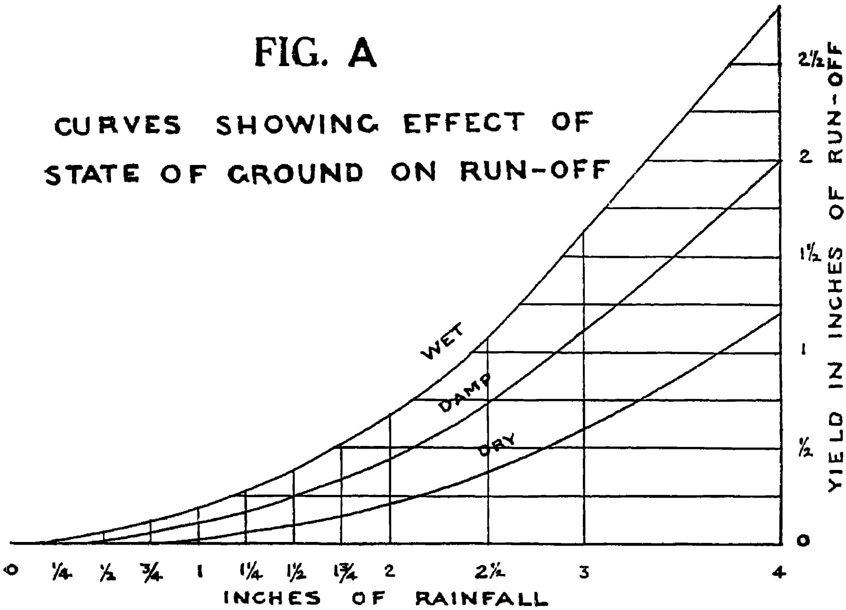


TABLE OF DAILY RUNS-OFF.

1	2		3		4	
Daily Rainfall in inches	Run-off Percentage and Yield when the original state of the ground was					
	Dry		Damp		Wet	
	Percentage	Yield in inches	Percentage	Yield in inches	Percentage	Yield in inches
0·25 .	—	—	—	—	8	0·02
0·50 .	—	—	6	0·03	12	0·06
0·75 .	—	—	8	0·06	16	0·12
1·00 .	3	0·03	11	0·11	18	0·18
1·25 .	5	0·06	14	0·17	22	0·28
1·50	6	0·09	16	0·24	25	0·38
1·75	8	0·15	19	0·33	30	0·52
2·00 .	10	0·21	22	0·44	34	0·67
2·50 .	15	0·38	29	0·73	43	1·08
3·00 .	20	0·60	37	1·12	55	1·65
4·00	30	1·20	50	2·00	70	2·80

In this table, the percentages of run-off from wet surfaces, compared with those from dry surfaces, are proportionately greater for light falls than they are for heavy falls. In the case of the latter class, the surfaces originally dry will have become saturated before the heavy falls have ceased, and the conditions producing run-off from both states of surface will thus be more alike. Compared with each other, the percentages of run-off from each state of the ground increase directly with the intensity of the rainfall. The defect of the table is that it takes into account the daily, and not the hourly, intensity of the rainfall. However, as explained in paragraph 17, p. 26, it is the former which is generally observed.

19. Estimation of the Run-off from the Daily Rainfall.—The proper way to estimate the run-off, in the absence of observations of it, will be to proceed on the lines of the above table, and, if a few discharge observations have been taken, to construct the table with reference to them. If the rainfall for a series of years is thus dealt with, a very fair idea can be obtained of the sufficiency, or not, of a catchment. Should it thus be ascertained that during years of average rainfall the tank will probably fill, and that in years of scanty rainfall, not amounting to absolute deficiency, there will be a fair replenishment, the catchment may be considered a sufficiently large one. As such a determination will be based on estimates of the percentages of run-off, care should be taken not to over-estimate them.

Another way of estimating the sufficiency of a catchment is to take, for a series of years, the average annual amount of the heavy falls of rain, each exceeding one inch (excluding years both of excessive and of

greatly deficient precipitation, and falls which occurred when the ground was dry), and to apply a general percentage for run-off to this. If the result shows that the catchment is likely to produce the required replenishment, it may be accepted as sufficiently large. This method is less laborious than the former one, but, in proportion to its diminished detail, is less likely to be accurate.

Such estimates are but approximations, and should be resorted to only in the absence of actual discharge observations. It is a matter of the first importance to have such observations for as many years as possible, and, therefore, whenever practicable a large scheme should not be proceeded with until a record of at least ten years has been obtained. Of course, where they exist, the results of observations for other similar and similarly situated catchments may be utilised to diminish the period of examination. Even when this can be done, it is advisable for large schemes to institute comparative, concurrent observations of the run-off of the catchments to be dealt with, with those for which observations have already been recorded so as to determine what factor should be applied to the previous results of the latter to enable them to be utilised for the former.

IV. The Storage Capacity of Reservoirs.

20. Proportion of the Storage Capacity to the Yield from the Catchment.—It is necessary to determine the storage capacity of a reservoir by reference chiefly to the run-off from the catchment which can be impounded, as, in the large majority of cases, the area which can be commanded will be in excess of

that which can be irrigated by the scheme. Where this is not the case and the area of irrigable land is limited, its extent has also to be taken into account in the manner described in paragraph 26, p. 45.

It will be best to limit the storage capacity to that which can be replenished in an ordinary year, as it is desirable in irrigation from reservoirs that the irrigated area should not greatly fluctuate. If that capacity is made much greater than this, it will be only in good years that the reservoir will fill, and, in all other years, there will be a certain amount of unproductive capital expenditure due to the increased size of the works beyond the requirements of the then storage. A small increase of storage in excess of the average amount of replenishment, *i.e.*, one not exceeding 10 per cent., is, however, desirable to compensate for the reduction of capacity that will occur by the silting up of the tank basin. The future need not be much anticipated in this respect, as the original amount of storage, when greatly reduced by silting, may be restored by raising the full-supply level and the height of the dam above it. A slight increase of storage may, however, be advisable to provide for a possible under-estimation of the run-off, and so as to take advantage of years of good replenishment.

If the storage capacity is originally made much less than the average replenishment, there may be a diminished rate of return on the capital expenditure on the work, as the rate of storage per million cubic feet usually decreases with an increase of storage. There will also be an increase in the proportion of silt deposited to storage impounded.

If monsoon irrigation is to be practised under a reservoir, the quantity of water required for it should

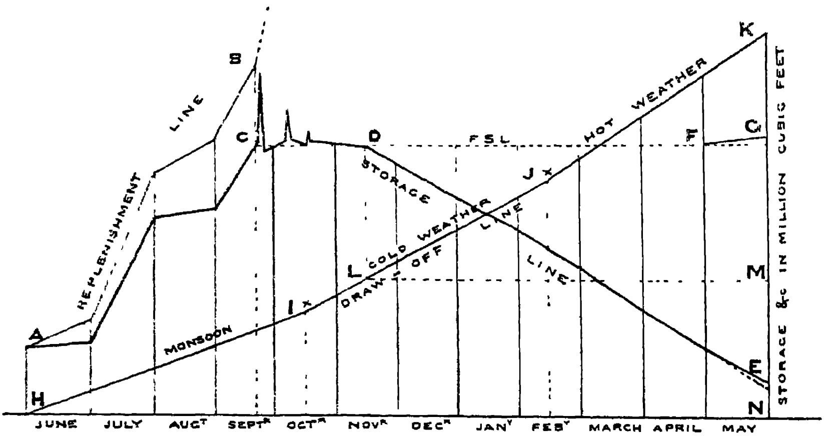
be taken into account. The catchment will in this case produce a certain amount of replenishment of which part will be expended during the monsoon, and the size of the reservoir should therefore be limited so as to store only the remainder.

20B. Determination of Sufficiency of Catchment.—The best way to estimate the sufficiency of a catchment is to proceed as described below. First, the full-storage capacity of the reservoir should be assumed and also its initial storage contents at the beginning of the first monsoon considered. A tabular statement should then be prepared in the first column of which should be entered the gross yield, or replenishment, during each month of a single year, which should be calculated by applying the tabular statement of daily runs-off (para. 18, p. 26) to the actual rainfall gauged. Other columns should similarly give for each month the loss due to water run to waste, the loss due to evaporation and absorption, the consumption by irrigation and the total of these amounts. The last column should show the figures of net storage impounded, which will be arrived at by deducting the total loss from the gross yield. The results of one year having thus been tabulated, those of other years should similarly be recorded. If at the end of the fair weather the net storage is generally a *minus* figure, the full-storage capacity of the reservoir should be increased accordingly if the calculated yield shows this may be done: if it may not, the draw-off, *i.e.*, the irrigating capacity of the work, should be decreased correspondingly. If, however, the final net storage is generally a large *plus* figure, the irrigating capacity of the project should be increased if the area under command permits.

From the annual tabular statement a storage diagram (Fig. B.)[1] showing the net storage at the end of each month can be plotted. In this diagram the ordinates represent the calculated replenishment, draw-off (including all losses and consumption), and balance storage in million cubic feet at the end of each month, and the abscissae the months themselves. The replenishment line, AB, and the

FIG. B

STORAGE DIAGRAM



draw-off line, HIJK, are first plotted, and the storage line, ACDE, is deduced from them, its ordinates being equal to the difference between those of the other lines. The line AB and the portion AC represent what took place when the reservoir was filling until it first attained full-supply level; the portion CD

[1] See also "Minutes of Proceedings, Inst C.E.," Vol. lxxi., p 270, and "Public Water Supplies," by Turneaure and Russell, 1st edn., p. 313.

extends for the period from when the waste-weir came into play until it ceased to flow; and the portion DE indicates the storage capacities during the rest of the year. To enable this last portion to be plotted, horizontal lines are drawn through D and L (where the vertical through D cuts the line HIJK), and the storage ordinates below DFG are made equal to the draw-off ones above LM. FG represents a small increment of storage due to hot-weather storms. If the reservoir does not fill, the point D for its diagram would be that of the highest storage then attained. If it is desired to ascertain the amount of yield run to waste over the waste-weir, the line AB would have to be extended to D^1 vertically above D_1 and DD^1 (less the draw-off during the period concerned) would represent that amount, and would indicate by how much the full-supply storage could be increased by enlarging the reservoir. To establish the amount of storage desirable, the diagrams of several representative years, and not only that of a single one, would, of course, have to be considered.

21. Calculation of Storage Capacity.—The calculation of the storage contents of a reservoir is made by summing up the contents between each of its contours. The formula to be used for calculating these contents is:

$$Q = \frac{H}{3}(A + a + \sqrt{A \times a}),$$

where Q is the storage in cubic feet;

A, the area of a contour in square feet;

a, that of the adjacent contour in square feet; and

H, the vertical distance between the contours in feet.

Although it is unnecessary that contours should be surveyed at small vertical distances apart, the calculation of the storage will be more exact if such contours

are taken into account. The general practice is to survey them at vertical differences of 10 feet and by calculation to interpolate others at 1 foot intervals. To determine the areas of these interpolated contours, it is usual to take the square roots of the surveyed contours and to consider that the square roots of the interpolated ones vary in exact proportion to their vertical distances apart from the others. A calculation of the capacity of a reservoir is given in Appendix 17, p. 382, and this shows the method followed in ascertaining the areas of the interpolated contours and the storage contents between them.

In determining the total storage capacity of a reservoir, it is not usual to add the further capacity due to the excavation of the borrow pits which furnish material for the dam. The contents of these will be very small relatively to the amount of natural storage, and, in respect to the capacity available for irrigation, will be still smaller, as a large proportion of them will be below outlet sill level.

From this "gross storage capacity" of the reservoir has to be deducted the storage below outlet sill level, as this cannot be utilised for irrigation except by pumping, which, in practically all cases, is too expensive a measure to which to resort. The balance storage—the quantity above outlet sill level—is known as the "gross available capacity," and this is all that can be taken into account when determining the irrigating capability of the reservoir. As it is not likely in most cases that the storage will fall below outlet sill level before the commencement of the monsoon, it will generally be quite safe to take only this "gross available capacity" into account when considering the total amount of probable replenish-

ment. After the reservoir has been drawn down to a little above the full-supply level of the canal from it, the rate of discharge of the canal will necessarily be reduced, but then, anyhow, economy in utilising that discharge will have to be practised. As a last resource the water from the reservoir can be lifted to supply the canal, and hence all the contents of the tank down to outlet sill level may be taken as "gross available capacity."

From the "gross available capacity" has to be deducted an allowance for loss by evaporation and absorption so as to arrive at the "net available capacity": it is this smallest quantity which determines the irrigating capacity of the storage.

22. Loss by Evaporation and Absorption in Reservoirs.—The total loss from evaporation and absorption can be calculated by noting the total fall of the reservoir surface, and by deducting from the loss of storage thus indicated the actual, measured amount utilised for irrigation. The separate estimation of the amount of loss due to each of these causes is not so easy to determine: it is therefore usual to take together the two kinds of loss.

The loss due to evaporation in an experimental cistern can be easily gauged, but this will be due solely to the effect of the temperature of the air on the surface of the water and on the sides of the cistern, which sides, of course, introduce an artificial condition, increasing evaporation. In the case of a large body of water, such as a reservoir, the amount of evaporation, however, greatly depends also upon the drying effect of wind (producing waves and spray), which will not equally affect the contents of a small cistern, and cannot therefore be accurately ascertained by its use.

It may, however, be said that these two causes of error in observation tend to balance each other. In respect to evaporation alone, the loss will be in direct ratio to the dryness and velocity of the air and to the area of the water surface, and will be in inverse ratio to the depth, as the temperature of shallow water is always higher in the tropics than that of deep water.

The loss due to absorption alone depends upon the nature of the bed of the reservoir basin and the depth of the storage. It will increase with the porosity of the underlying strata, and with the pressure due to the depth of water. In ordinary cases, and more especially after some years when the bed of the reservoir has become waterproofed by the deposit of silt, the loss from absorption will generally be much less than that from evaporation; hence deep reservoirs, which present a comparatively small surface, will usually suffer less total loss on these accounts than will shallow ones. In the case of porous basins the loss by absorption will to some extent be diminished by springs feeding the reservoir from its margin, which will frequently be equally porous.

In the loss by absorption is included the loss by leakage below the dam, to reduce which a puddle trench is formed under the embankment. This loss should be comparatively very small.

The total loss will depend upon the time that these causes act on the storage, and will vary with the season. During an ordinary monsoon, as the air is then charged with moisture and is fairly cool, there would be comparatively little evaporation, but for the fact that then the wind for many days blows with violence and thus causes a good deal of it to take place. In this period, however, the rainfall which falls on the

water surface of the reservoir will be wholly impounded, and this should fully make up for the evaporation which then takes place, as in the run-off calculations it is not usual to consider the area of the tank separately from that of the rest of the catchment, from which the run-off is only a fraction of the rainfall.

In the cold weather evaporation will not be great, as, not only will the temperature then be low, but there will generally be an absence of high winds. In the hot weather, when the temperature rises, and hot, dry winds of considerable velocity may be expected, the vertical loss will be at a maximum, and may amount to as much as 0·4 [1] inch a day, but the surface exposed to them will be at a minimum.

In regard to absorption the loss will be greatest when the reservoir has filled in the monsoon, less as its storage diminishes in the cold weather, and least in the hot weather when its contents are small.

23. Observations of Loss by Evaporation and Absorption in Reservoirs.—The table on p. 38 records some observations of the amount of loss that have been made.

In Madras the S.W. monsoon (June to September) does not produce much rainfall and the temperature is then high; there most of the rainfall occurs during the N.E. monsoon (October to January) which takes place in the cold weather—that season is, however, much warmer there than it is in the more northerly parts of India.

The average daily loss throughout the year was found to be 0·28 inch and the annual loss, 8·39 feet; this large amount seems to indicate that there was

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[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 65.

much loss by absorption. The heaviest rainfall is in October and November, when it varies from 10 to 13 inches each month.

[1] AVERAGE DAILY LOSS IN THE RED HILL TANK, NEAR MADRAS.

S W Monsoon.			Cold Weather and N E Monsoon			Hot Weather		
Month	No of years observed	Loss in inches	Month	No of years observed.	Loss in inches	Month	No of years observed	Loss in inches.
July ..	4	0·33	November	1	0·27	March .	5	0·26
August ..	5	0·32	December	2	0·13	April .	5	0·30
September.	1	0·38	January ..	4	0·24	May .	5	0·37
October ..	1	0·27	February	2	0·24	June ..	4	0·30
Average	..	0·33	..	..	0·22	..	.	0·31

In the Ekruk tank, near Sholapur, Bombay Presidency, the loss from evaporation and absorption from April 17th to May 29th, the hottest part of the year, averaged 0·384 inch a day, while the similar loss, increased by some leakage, during November to March averaged 0·232 inch a day.

The loss [2] in tanks in Rajputana is given by Mr. W. W. Culcheth as follows :—

Season.	Average daily loss in inches		
	Evaporation.	Absorption.	Total
During the irrigation season, October to March	0·15	0·05	0·20
During the hot season, April to June	0·29	0·17	0·46
During the rainy season, July to September	0·21	0·20	0·41
Average of the year . . .	0·20	0·12	0·32

[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 64.
[2] *Ibid*, p. 64.

The soil was porous, and yet the loss from absorption was considerably less than that from evaporation ; the former loss is irregular in amount, and it was probably difficult, as explained before, to measure it exactly. The total loss in the year is given as 9·77 feet, of which it is said 6·11 feet was from evaporation and 3·66 feet was from absorption. These figures are unusually high.

In the Páshán tank,[1] near Poona, the average daily loss, from evaporation alone, was in inches :—

October	November	December.	January	February,	March	April	May.
0·25	0·19	0·14	0·17	0·14	0·17	0·27	0·38

During these eight fair-weather months, the total loss was 4·33 feet.

The usual allowance made in Bombay projects for all losses is 4 feet, measured on the mean area of the tank, although, in some cases, it has been taken as low as 3 feet, and, in others, as high as 7 feet (Appx. 1, p. 347A). As the surface contour areas of a reservoir in Bombay are largest during the monsoon when evaporation is moderate, and are smallest during the hot weather when that is great, the estimate of the total amount of loss during the year, which is arrived at from an assumed vertical depth on the mean area of a reservoir, is equal to a greater vertical depth of evaporation on the actual contour areas exposed during the different seasons. Moreover, as stated in paragraph 22, p. 37, credit is not taken for the extra amount of the yield of the rain falling on the surface of the reservoir, and this may be considered as a reduction of so much loss by evaporation.

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[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 65.

If it is desired to make a detailed estimate of the amount of evaporation the following may be taken as the average monthly loss during the principal seasons of the year :—

Cold weather	. 3 ins.	—total for four months	= 12 ins.
Hot weather	. 10 ins.	,, ,, ,,	= 40 ins.
Monsoon	. 5 ins.	,, ,, ,,	= 20 ins.
		Total for year .	6 feet, or 72 ins.

It will be seen from the above that for irrigation schemes it is not economical to store water for two years, as the loss from natural causes during so long a period will amount to a very large proportion of the total storage. It would be far better, both for Government and the cultivators, to utilise in one season the full supply available, so as to sustain only one year's loss by evaporation and absorption. The money value of the quantity of water which would thus be lost by storage for irrigation for a second year is considerable, and such storage would, moreover, be rendered unnecessary (at least temporarily) by a good replenishment during the second year. Storage to tide over a second year with bad replenishment is justifiable only in the case of a reservoir for the waterworks of a town, as for that a continuous supply must be secured even at the increased cost of a capacity larger than is required for the wants of a single year.

24. **Observations of Loss by Evaporation and Absorption in Canals.**[1]—As feed channels have to be made in connection with some reservoirs, the following results of the observations of loss on certain canals are given :—

[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 67.

(*a*) The Háthmathi Canal, Ahmedabad district, Bombay, with an ordinary discharge varying from 20 to 100 and a maximum discharge of 191 cusecs, is estimated to lose 50 per cent. of its discharge in the first 10 miles of its course.

(*b*) The Nirá Canal, in the Poona district, Bombay, with a head discharge of 455 cusecs, and a length of 101 miles, has been gauged to lose 1 cusec a mile, or, altogether, 22 per cent. of the initial supply.

(*c*) The Muthá Canal, also in the Poona district, with a maximum discharge of 412 cusecs, loses from 0·8 to 0·9 cusec a mile.

(*d*) The Patna Canal, Bengal, having a bed-width of 69 feet and an average depth of about 6 feet, had a gauged loss of 40 cusecs in a length of 7 miles when it was new. The rate of loss has decreased greatly since the canal has silted.

(*e*) On the Bári Doab Canal, in the Punjab, after the canal had been open for from sixteen to eighteen years, the following gaugings were made :—

Section of Canal	Date of Observation.	Upper Gauge.		Lower Gauge		Loss		
		Mean velocity, ft per sec.	Discharge in cusecs.	Mean velocity, ft per sec	Discharge in cusecs	Discharge in cusecs	Percentage. Total	Percentage. Per mile
Between Madhopur and Chandeki, 52 miles apart	March 1st and 2nd	—	2,009	2·68	1,728	281	14·0	0 27
	May 26th and 27th	4·44	2,142	2 93	1,874	268	12·5	0·24
	May 28th and 29th	4·33	2,036	2 79	1,789	247	12·1	0 23
	June 9th and 10th	4·44	2,165	2 77	1,874	291	13·5	0 26
Between Hibban and Gandian, 63 miles apart.	July 31st	—	289 4	—	243 2	46·2	15·9	0 25
	August 2nd	—	384 3	—	298·4	85 9	22·4	0·35

When such observations are to be taken, a canal should be made to have a constant discharge for a few

days before and during the time of the gauging, so that its perimeter may be thoroughly wetted (in order that excess absorption may be prevented), and during that time all distributaries should be closed and all excess leakage should be cut off.

As in reservoirs so in canals, the total loss is due to the combined effects of evaporation and absorption. The amount evaporated will depend upon the surface area exposed, the temperature and dryness of the air, and the velocity of the wind. The loss by absorption will vary with the wetted perimeter, the pressure depth, and the porosity of the strata passed through, and will usually diminish as the canal gradually puddles its sides and bed by the deposit of silt on them. The usual allowance in Northern India for the total loss is 8 cusecs per million square feet of wetted perimeter; the surface area of the water is there not taken into account.

24B. Reservoirs in Series.—(*a*) It is generally better to store the same amount of water in one than in several reservoirs, as this will usually reduce the cost of storage, the supervision of the works, the chance of failure, and the loss by evaporation and absorption.

(*b*) Reservoirs should, if possible, not be "in series", that is, one below another, as the failure of an upper one may lead to the destruction of the lower ones, one after the other.

(*c*) Where reservoirs in series cannot be avoided, their size should be regulated so that the lower a reservoir is the greater is its capacity, in order that it may be able the better to absorb the flood resulting from the failure of an upper one.

(*d*) When along the course of a valley there are

several sites available for reservoirs, care should be taken (para. 43^A, p. 65) to ascertain which are the best and if a reservoir is constructed, that it will not interfere with the sites of other good ones.

(*e*) From an engineering point of view the head of a catchment should be dealt with first, and lower projects constructed in the order of their nearness to it. More knowledge of the yield-capacity of the whole catchment will thus be gained and the impounded water will first be utilized at the highest level practicable.

(See also para. 170, p. 228.)

25^B. Temporary Storages—Flood Regulation.—Reservoirs which do not impound water for prolonged supply may be classed as "temporary storages." They may be wanted for—(*a*) aiding permanent storage; (*b*) stands-by for town water supply, power or lighting schemes; or (*c*) flood regulation: they may be formed by earthen embankments or masonry dams. The first two classes are described generally in paragraph 9, p. 13, and paragraph 226, p. 326.

A flood-regulating reservoir is required on a large river, or a considerable tributary, when the uncontrolled floods thereof are causing extensive damage either to towns or agricultural tracts. Properly to regulate such floods the capacity of the reservoir should be so great that it will never attain high-flood level, for should it rise thereto, the subsequent high-flood discharge will pass through it with undiminished intensity, although its duration, and thus its power for damage, will be lessened. As in the previous cases, such a reservoir can, however, be reduced in size so as to store only the yield of a maximum flood, if its contents can be run off in the intervals between floods:

to enable this to be done the work should be provided with large outlet sluices giving considerable discharge.

If the reservoir is on the main stream and its water is not to be utilised, it may not be necessary to regulate the discharge from the outlet and then sluices need not be supplied, as the vents can be left constantly open. If, however, the reservoir is on a tributary stream, it may be desirable to impound its contents while the main river is in high flood, and to discharge its storage after that flood has decreased, and then sluices will have to be fixed. Similarly, sluices will be required if the storage is to be utilised for irrigation, etc. The outlet thus forms an important part of the whole design: it should be provided, when necessary, with large sluices. It is advisable, for structural reasons, to group these together and to place them at or near the stream course so as to maintain a definite scouring channel through the reservoir, as this will tend to diminish the deposit of silt on its bed. For an earthen dam the best type of outlet for this reason is that with its head wall in the centre line of the dam (para. 205, p. 296). For a masonry dam the outlet can be situated in the generally most suitable position, which will usually be above the river bed.

For an earthen dam a site for a sufficient waste-weir is essential; for a masonry dam such a site is less necessary, for failing it, the surplusage of floods can be passed over a lowered part of the crest.

25. Comparative Cost of Storage.—In comparing the relative costs ("works" charges only) of storage of different schemes, the rates of storage per million cubic feet of "gross available capacity," and not of "gross storage capacity," should be taken into account, because for the purposes of irrigation the storage

below outlet level is useless. A better comparison would result if also the amount of loss by evaporation and absorption were deducted from the available capacity to arrive at the "net available capacity" (para. 21, p. 35). This is, however, never done, because the amount of the deduction to be made is not known exactly but has to be estimated; moreover, it will vary with the way in which the storage is expended, being least when the contents of the reservoir are drawn on extensively in the early part of the season.

This comparison of rates of storage is a most valuable one, as from it can be seen at once if a scheme is likely to be a remunerative one. In Bombay, where the rate of earthwork varies from about Rs. 1..2..0 per 100 cft. in small dams to about Rs. 1..8..0 in large dams, the following would be considered fair rates of storage for—

	Mill. cft	Rs per mill. cft. gross available capacity
Small reservoirs storing less than ..	200	750
Medium reservoirs storing from	200 to 1,000	500
Large reservoirs storing more than ..	1,000	400 to 300

26. Estimate of the Storage required for a certain Irrigable Area.—In some cases there may be a definite area to be irrigated, either wholly by the storage impounded in a reservoir, or, partly, by the natural discharge of a stream, and, partly, by storage, and the amount of the storage required has to be determined. A typical calculation to determine this is given in Appendix 3, p. 350. In this case a full-supply storage is assumed at first and the effect of the draw-off from the reservoir is worked out. If this shows a deficiency of supply, the maximum amount of deficiency should be

added to the assumed storage, and to this allowances for evaporation and absorption and loss in transit down the feed channel should be added to arrive at the required amount of storage. If the reservoir with the storage originally assumed shows an excess supply, the amount of the excess should be deducted from it before adding the allowances mentioned above.

27. Estimate of Expenditure from Storage—Duration of Supply.—The consumption of water from a reservoir should be regulated as carefully as expenditure of money, for, otherwise, the storage may not suffice till the monsoon of the next year restores it, and the duties expected from the supply may not be realised. For this purpose an estimate of draw-off, etc., should be made, as given in the example in Appendix 4, p. 352, and as explained in the notes appended to it, and the calculated rate of draw-off should not be exceeded without prior sanction.

In Appendix 5, p. 354, is given another estimate of the duration of a water-supply storage, and the method of its calculation is explained therein.

28. Preliminary Estimate of the Cost of a Storage Reservoir.—Before a scheme is drawn up in detail, it is necessary to ascertain approximately if it will, or will not, be financially remunerative. The cost of a reservoir depends chiefly upon that of the dam, and this can be estimated roughly and quickly from the longitudinal section. To it have to be added the costs of the outlet, waste-weir and land compensation, which can be taken out either approximately and independently, or with reference to the rough estimate of the dam, or can be arrived at from the cost of similar existing works.

The following table shows the comparative total

costs of the subworks, etc., of twenty completed projects with earthen dams in the Bombay Presidency (Appx. 1, p. 347A). It will form a guide as to the amounts which the subworks of a contemplated project should cost.

1	2	3	4
Subwork	Total Cost in 20 Projects Rs	General Percentage Cost of Subwork of Total Cost	General Percentage Cost of Subwork of Cost of Dam
1. Dam	33,70,038	77·95	100·00
2. Outlet	2,33,473	5·40	6·93
3. Waste-Weir	4,08,654	9·45	12·12
4. Land Compensation .	3,11,218	7·20	9·23
5. Total	43,23,373	100·00	128·28

The total cost of the individual projects varied from Rs. 30,732 to Rs. 9, 43, 421 : 7 works cost under Rs. 1 lakh each ; 9 works, from Rs. 1 to Rs. 3 lakhs each ; and 4 works cost over Rs. 3 lakhs each.

In the table, col. 3 gives the general percentage cost of each subwork of the total cost of the projects, and col. 4, that of the total cost of the dams alone.

The capacity of the reservoir having been ascertained, the approximate rate of storage per million cubic feet can thus be found out. The rate due to the canal and distributing works will then have to be added, the total cost of these being deduced from that of existing projects. The approximate net return from one million cubic feet of storage can be estimated as shown in Appendix 7, p. 361, and it can then be seen if this is sufficient to produce a proper return on the expenditure.

V. Irrigation Considerations.

29. The Amount of Storage dependent upon the "Duty."—In order to ascertain the quantity of storage required in a reservoir to irrigate a certain area, or, *vice versâ*, the extent of the area which can be irrigated by a certain amount of storage, it is necessary to know first what is the irrigation "duty" of the different classes of crops. One way of defining "duty" is to take it as the acreage which can be brought to maturity by the constant flow, during the season the crop is on the ground, of one cubic foot of water a second. By this definition a "duty" of 100 acres means that one cubic foot a second will suffice for maturing that area of crop. The definition refers only to the rate of irrigation and not to the total quantity of water required (as it does not take the element of time directly into consideration) and it assumes that the rate of supply is constant throughout the whole period of cultivation, which, however, is not the case.

Another way of reckoning the "duty" is to consider the average depth of water poured over the area during the whole irrigation season: in Northern India this depth is called "delta," or Δ, and the efficiency of irrigation thus varies inversely with Δ. A similar method is to express the duty in terms of "acre feet"; an "acre foot" is the volume of water standing one foot deep on an acre of land. The number of "acre feet" utilised in irrigation is the area in acres irrigated multiplied by the average depth of the water poured on to it: this quantity in cubic feet is found by multiplying the number of "acre feet" by 43,560, the number of square feet in an acre (Appx. 25, p. 423).

A third way of stating the "duty" is with reference to the number of acres which can be irrigated by the storage of one million cubic feet of water. Thus a reservoir will have a "duty" of 4 acres per million cubic feet if the crops on that area can be brought to maturity by applying to them a million cubic feet of stored water (Appx. 7, p. 360).

The "duty," in whichever of the three ways it is reckoned, depends upon where the measurement of the water is taken, *i.e.*, at the source of supply or at the head of the irrigated area. It has been estimated that only between 50 and 60 per cent. of the water which enters a canal actually reaches the fields. The greater the "duty," the larger the irrigated area.

Appendix 6, Table I., p. 356, gives the duties which have been obtained during some non-famine years on certain irrigation works in India supplied by reservoirs and canals. It will be seen from this that the duties vary greatly, and that the main general result is that they are smaller on works dependent upon reservoirs than on those that are served by canals without artificial storage. This is due to the fact that the country in the neighbourhood of reservoirs is generally of a more irregular nature, and the area under irrigation by them is less concentrated, than is the case in the alluvial tracts irrigated from canals without storage.

Appendix 6, Table II., p. 357, gives the duties which have been obtained during some non-famine years on certain irrigation works in Bombay (Deccán). These also vary considerably owing to local peculiarities; the main general result is that the kharif duties are higher than the rabi ones; this is probably due to the rainfall, which is practically limited to the former season.

To determine the amount of storage required, it is best to ascertain the duties obtained from constructed works in the neighbourhood, but, where this information is not available, it will be necessary to estimate, or assume, them. As a general estimate it may be taken that under reservoirs the following will be the average duties when irrigation is fully established :—

Monsoon dry crops, 160 ; rabi crops, 120 ; perennial crops, 100 ; hot weather crops, 60 ; and rice, 40 acres per cusec (Appx. 7 (1), p. 360).

30. The Proportion of Irrigable Area to Culturable Area under Command.—Owing to the expense of, and soil exhaustion produced by, irrigation, it is not likely that the whole culturable area commanded by a reservoir will be irrigated. Appendix 6, Table III., p. 358, shows the actual results obtained on irrigation works in the Bombay Presidency (Deccán). During the latest years therein recorded the percentage of the irrigated to the culturable area under command was 33·9, but, as on these works the land at the tail of the systems had generally not been brought under command, and, as the "duty" there will be less than it will be at the head, (owing to greater loss in transit), it will be safer to assume that not more than one-quarter of the culturable land under command will eventually be irrigated annually by a reservoir.

On the great canal systems of Northern India, in certain tracts where water-logging and consequent salt efflorescence have to be prevented, the percentage of irrigated area has to be restricted, rather than increased. It has been decided that, in the area served by the Chenáb Canal,[1] as long as the spring level is

[1] "Recent Developments in Punjab Irrigation," by Sidney Preston, C.I.E., "Journal of the Society of Arts," Vol. i., No. 2584, May 30th, 1902, p. 607.

more than 40 feet below the surface, the limit of the percentage of irrigated to culturable area may be 50, but, as the water level rises the limit is to be reduced, until when the subsoil saturation plane is within 10 to 15 feet of the surface, irrigation is to be stopped altogether. On lands irrigated by reservoirs where the surface of the ground is generally not uniform in level and is intersected by numerous well-defined drainage lines, such water-logging is rarely to be feared or guarded against, but still cases of it have there occurred. The remedy to be applied, when necessary, is the proper drainage of the area affected.

31. The Proportions of different Crops under Irrigation.—To estimate the revenue probable from a work it will also be necessary to take into account the proportions in which the different crops are likely to be irrigated, as the quantities of water required for them and the assessments on them vary. The factors which determine these proportions are :—

(*a*) The suitability of the soil and climate ;
(*b*) The sufficiency of the water supply ;
(*c*) The market prices of the produce.

Of these (*a*) are fairly constant ; (*b*) should, as far as practicable, be made constant by good arrangements ; but (*c*) is beyond local control, and variations due to it must be accepted as not preventable.

Appendix 6, Table IV., p. 359, shows the average results obtained during certain years on certain works in Bombay (Deccán). It is best to take into account the results of neighbouring works situated under like conditions to the project under preparation, but, where such works do not exist, it will be fairly correct

to assume that the following percentages of crops will be irrigated :—

Perennial and hot weather	20	Total 100
Rabi	20	
Monsoon dry	40	
Eight months	20	

(Appx. 6, Table IVA., p. 359.)

32. The Rotation of Crops.—The above percentages indicate that a system of quinquennial rotation is practised under the recently established Bombay irrigation tanks, namely two years of monsoon dry to one year of each of the other classes of crops. This is the lowest form of rotation, while triennial rotation—perennial, eight months, and rabi crops—is the highest. Rotation is necessary to secure restoration of fertility to the soil, which would, without it, and artificial help from manures, soon become exhausted if constantly irrigated by stored water, as this is practically devoid of fertilising silt. Different crops remove different constituents of the soil from it, but these by the action of the weather are wholly, or partly, reproduced in the ground during the period of rotation, which thus has an effect similar to that of a fallow. The rotation should, however, be as high as possible, for the revenue and value of the produce chiefly depend upon the proportion of the perennial crops to the whole area under irrigation (Appx. 7 (3), p. 361).

VI. Reservoir Basins.

33. The Proportion of the Submerged Culturable Area to the Area Irrigable by the Project.—The storage of water in a reservoir involves the submergence

of the land in its basin, and this, in the majority of cases, renders it unculturable. If the catchment area is rocky, the bed of the reservoir will in time become covered with a layer of sandy material, which is unsuited to cultivation. If, however, the drainage area is of soil, the bed will eventually be overlaid by fertile silt, which may render it better fitted for agriculture than it was originally ; but, even in this case, it will at first be too damp to plough and may subsequently cake in drying, so that crops cannot be grown on it. Moreover, the bed may not be laid dry until the cultivating season has passed. Generally only shallow tanks, the contents of which are expended during the monsoon, can have their beds cultivated, and that can be done chiefly in the rabi season. It may thus be said that the construction of a reservoir usually throws the submerged area permanently out of cultivation: much of this owing to its situation in a valley may originally have been very fertile. It is therefore necessary that this area, when culturable, should be as limited in extent as possible, for the land to be served by the work may not always be irrigated, and, if the submerged culturable area bears a large proportion to it, there may not be, in a series of years, a substantial increase of cultivation compared with that which existed before the construction of the project. This argument, of course, applies only to culturable land, or land which can be cultivated, as waste lands, except so far as they are useful for grazing, need not be taken into account in this connection. Moreover, in regard to the culturable land under command, it has to be remembered that its produce will be largely increased by irrigation. Speaking generally, the culturable area in a reservoir basin

should not exceed one-fifth of that which can be irrigated by the work.

It is desirable, especially in newly settled countries, to obtain favourable basins for storage before the areas are occupied and become too valuable for acquisition. For this reason deep reservoirs, which occupy little land compared with the quantity of their storage, are to be preferred to shallow ones. The former have the added advantage that, as they have a comparatively small surface area, the loss by evaporation from them will be at a minimum. In this respect the best storage basins are those which open out immediately behind the dam, as this part of their area has the greatest depth of water over it. The slopes of the country near the dam should preferably be flat ones, but it is not of so much importance (until the storage is to be increased) that those at the head of the reservoir should be gentle, as there the maximum depth of water will be shallow.

34. The Retentiveness of the Reservoir Basin.—It is necessary for economical storage that the bed of the basin should be of a retentive nature, so that the loss by absorption may be as small as possible. It is true that the longer the tank is in use, the more will it be water-proofed by the deposit of silt on its bed; but this silt is itself naturally porous, owing to its recent deposition, involving want of consolidation by pressure, and it is therefore best to start with naturally retentive substrata. Moreover, if a large area of the tank bed is annually laid dry, the overlying silt will absorb an appreciable amount of storage, which will be a final loss unless the reservoir overflows in the monsoon, as then it will restore that loss. The most retentive basins are those with deep, clayey, alluvial

beds, and the least retentive are those with vertical and fissured strata. When the fissures are of great length and size, and particularly when they are in soluble rock, such as dolomite, they may lead to much loss of water and possible danger to the works, and may thus make their construction not advisable. Careful geological examination of the reservoir basin in such a case is therefore necessary before deciding to carry out the project originally contemplated.

35. The Extent and the Utilisation of the Land to be acquired.—When land has to be acquired for a reservoir, it should be taken up at least to high-flood level, as otherwise the submergence during floods of marginal crops may lead to their destruction and to claims for the payment of compensation. In fact, if the land is cheap, it should be acquired originally to a slightly higher level to permit of the future increase of the storage being obtained without having subsequently to pay extravagantly for the marginal land, which may, by that time, have become increased in value, either from natural circumstances, or through advantage being taken by its proprietors of the necessity for its purchase. The limits of the acquired land should at once be permanently marked by boundary stones, the positions of which with reference to the ordinary field survey marks should be off-setted and recorded on the maps and in tabular statements. Such limits should be in long straight lines, somewhat external to the contour to be acquired.

In order to render the acquired marginal land as productive as possible, it should be planted out at once with trees, as the young plantations can best be watched while the construction of the works is in progress. Bamboos and trees which flourish in damp

situations should be planted in the zone next full-supply level, and fruit-bearing trees, which will produce an annual revenue, should be reared in solid blocks near the dam, where they can most easily be guarded. The rest of the area should be devoted to less valuable trees, which can be cut down periodically for timber or fuel.

Reservoir irrigation exhausts the land very much, as the supply from storage is nearly clear, and devoid of most of the fertilising silt which is suspended in the water of canals fed by perennial rivers flowing through alluvial soils. In order to develop such irrigation it is necessary that there should be a plentiful supply of manure, and, as in India manure is much used for fuel, it is most desirable to establish fuel plantations to set this artificial soil-stimulant free for agricultural purposes. The reservoir plantations recommended will serve usefully for this purpose. Other plantations can be formed with advantage on waste lands commanded by the canal.

VII. The Silting of Reservoir Beds.

36. The Necessity and the Methods for the Reduction of Silt Deposit.—As noted in paragraph 34, the silting of reservoir beds has the advantage of making them more retentive, but it has also the disadvantage of reducing the storage capacity, which is a result far outweighing the former one. It is not economically practicable to remove this silt by any artificial means, as the cost of storage is very small compared with that of any system of excavation. As, under ordinary conditions, the water which enters a reservoir will precipitate practically all its silt in the basin, it is

desirable to diminish the accumulation of silt by choosing a rocky, insoluble, grass- or forest-clad catchment, which will not produce much deposit, and one with a certain replenishment, which will not necessitate an increased catchment area to insure full storage in bad years.

The most heavily silt-laden floods are those which come early in the monsoon, as they carry down with them the soil which has been desiccated and loosened during the preceding fair season. The most effectual way of dealing with the silt difficulty is to pass these early discharges out of the reservoir as rapidly as possible by means of outlets with large discharging power. The amount of replenishment due to these floods can, however, be foregone only in cases where the subsequent run-off is sufficient to ensure the filling of the reservoir. Such reduction of silt can therefore best be effected when it is necessary to impound only the clear-water flow subsequent to the close of the monsoon: an example of this treatment is furnished by the large Assuán reservoir on the Nile.[1]

Probably in India the simplest method of removing part of the silt is to plough the margin of the reservoir, exposed by the decrease of its level, as early as possible so as to loosen and desiccate its surface. The scouring action of subsequent rain will carry some of this silt into the pool at the bottom of the tank basin, and it can thence be discharged through the outlet. With a similar object in view, the deposit for a short distance from this pool may also be moved closer to it by means of scrapers.

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[1] "Minutes of Proceedings, Inst. C.E.," Vol. clii., Paper No. 3361.

37. The Classification of Silt.—The deposit in reservoirs may be divided into three classes :—

1. Heavy detritus, consisting of stones, pebbles, and large sand which is at once dropped ;
2. Heavy silt, consisting of fine sand and coarse earthy matter which is precipitated in a few days ;
3. Fine silt, consisting of flocculent earthy matter which is held in suspension for many days.

The first class is deposited directly the velocity o the inflow is checked by its entry into the reservoi pool, and will thus be found along the beds of th feeding streams, and particularly at the head of th reservoir. If the outlet is kept open for some time i the early part of the monsoon and is designed then t give a large discharge, the deposits of the previou years may gradually be moved forward towards th dam, and a part of them may eventually be swep through the outlet, but the proportion thus dispose of will generally be small compared with the tot quantity originally trapped by the storage.

The second class, being originally distribute uniformly throughout the contents of the reservoi will be precipitated in direct proportion to the dept of the storage, and it will thus be thickest in the dee parts of the reservoir nearest the dam, and in the mo favourable position for being passed out through th outlet. Being, however, of a tenacious and compa nature, it will resist removal unless loosened artificiall say, by ploughing, as mentioned above.

The third class, similarly to the second one b more slowly, will gradually be precipitated, and chief near the dam, and can similarly be dealt with.] both these classes the amount of silt which can]

removed by being passed out through the outlet will be only a small portion of that which has been carried into the storage. As the water in the reservoir will be in a state of quiescence for several months, practically most of the silt which enters that will be deposited in it: only the light sediment which would be held in suspension for some days will be carried over the waste-weir, or through the outlet when those are in flow.

The loss of storage by silting should not exceed annually $\frac{1}{1500}$ of the full-supply contents of a reservoir having a catchment selected to avoid great denudation. It may amount to much more if the gathering ground has a highly soluble surface. The Val del Inferno dam, 115 feet high, has silted to its crest.

38. The Proportion of Silt to Water.—The total quantity of silt brought into a reservoir will depend upon the nature and yield of its catchment. The table[1] below gives the results of some observations of the weight of silt brought down by certain rivers.

These large rivers flow through fertile areas and consequently carry much silt. Generally, reservoirs will be situated in hilly and less fertile tracts, and, in their case, the weight of silt will probably be less, and might on an average be taken as $\frac{1}{1000}$ of the weight of the water.

The silt when first deposited will be of a light, bulky nature, but, in course of time, will become more compact, and thus will occupy less volume, and eventually will attain a density nearly equal to that of ordinary soil. Taking the weight of soil as about 100 lbs. per cubic foot, the final bulk of the silt will be

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[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 33.

about two-thirds of that of the volume of water to which it was originally equal, and thus the volume of silt deposit, compared with that of the water in which it was suspended, will be about two-thirds of the proportion by weight.

	1	2	3	4	5
	River	Experimenter	Approximate Velocity, ft per second	Proportion of Silt to Water by Weight	Remarks
1	Mississippi .	Humfreys and Abbot	4	$\frac{1}{572}$	
2.	Rhône .	Gorsse and Subours	8	$\frac{1}{45}$	
3.	Po .	Lombardini	—	$\frac{1}{300}$	
4	Vistula	Spittel	10	$\frac{1}{48}$	River partly frozen.
5	Rhine. .	Hartsoeker	—	$\frac{1}{100}$	
6.	Nile .	Letheby	3	$\frac{1}{668}$	Equal parts of sand and mud.
7.	Rio Grande [1]	Anson Mills	10	$\frac{1}{290}$	By volume. Average of 118 samples
8	Indus .	Tremenheere .	5	$\frac{1}{237}$	Equal parts of sand and mud.
9.	Ganges .	Medlicott	10	$\frac{1}{760}$	At head of Ganges Canal

39. The Determination by Cross-sections of the Amount of Silt deposited.—There are three ways in which the amount of silt which is being deposited in a tank may be determined, by :—

1. Section lines across the reservoir bed ;
2. Silt boxes on the reservoir bed ;
3. Measurement of the inflow into and outflow from the reservoir.

In the first system, cross-section lines are ranged at intervals across the basin and have their ends permanently marked by masonry pillars about 5 feet high, so as to be visible from each other, and with their distinctive numbers and levels engraved on their capstones. Where it crosses the originally surveyed 10 feet contours each line is marked with header-stones, also engraved with their distinctive numbers and having their surfaces at original ground level.

[1] Schuyler's "Reservoirs," p. 361.

Iron index rods projecting 3 feet above ground level are fixed close to the headers so that their positions may be readily ascertained even after their tops are covered by silt. The original sections are plotted, and they are relevelled and replotted at intervals of a few years. Although this system will give the exact depths of deposit varying from point to point on the sections, it has the objections that it will take several years to ascertain any great difference of level, during which the record may be lost, and also that the number of lines has necessarily to be small, so that the total deposit over the whole bed may not be correctly ascertainable from them. In order to avoid an accumulation of errors of observation, each section taken at intervals of time should be compared with the original one, and not with the one last levelled on the same line.

40. The Determination by Silt Boxes of the Amount of Silt deposited.—In the second system, wrought-iron boxes, about 2 feet square and 1 foot high with closed bottoms and open tops are placed at intervals along section lines ranged across the basin. Each box has at top a light frame of inclined diagonal rods from the apex of which is fastened a substantial chain by which the box can be lifted, and to the other end of this is fixed a light wire attached to a large float, so that the position of the box can be ascertained when the reservoir is full. When the boxes have to be examined, they are carefully lifted and the contained silt is either measured or weighed. The objections to this system are :—the floats may break loose in stormy weather; the boxes are difficult to lift when in deep water, and, possibly, their contents may be upset while they are being lifted; and the number of lines and of

boxes in them have to be too few to determine accurately the total amount of deposit over the whole bed of the tank.

41. The Determination by Observations of Inflow and Outflow of the Amount of Silt deposited.—In the third system, samples of the inflowing water are taken, daily when the rate of inflow is steady and at more frequent intervals when it varies, and the proportions of silt to water, either by volume or weight, are ascertained by passing the whole samples through filter papers and collecting the silt on these. Until the waste-weir comes into action, the increase in the contents of the tank can be readily ascertained from the tank capacity table. While the waste-weir is flowing, samples of its discharge have similarly to be taken, the amount of that discharge has to be calculated, and the amount of silt carried away can then be determined and deducted from that estimated as brought into the tank. By this system, a very fair approximation to the actual amount of silt deposited can be obtained, but the way in which the deposit is distributed over the bed cannot thus be ascertained; this is, however, not a matter of much importance. The objections to this system are that it involves a large series of observations; that the samples of water taken may be not truly representative of the whole inflow and outflow; and especially, that the amount of heavy detritus deposited at the heads of the river and of its tributaries within the basin are not measurable by it. Still, it affords the quickest and most exact form of silt measurement, and can be made more accurate by combining with it measurements on cross-sections of the heavy silt deposited at the heads of the principal inflowing streams.

42. The Level at which the Outlet Sill should be placed.—It does not require any system of silt measurement to prove that the amount of deposit from ordinary catchments is very considerable, as after a few years this is most perceptible, and, even in twenty years, in many cases has been sufficient seriously to diminish the storage contents of existing reservoirs. It is for this reason not desirable to place the outlet sill at a very low level, for, if this is done, the tank bed will soon silt up above it. As a rule it will be advisable to fix its level, so that the original storage below it will be about 10 per cent. of that of the whole reservoir. The original contents of twenty existing tanks in Bombay above and below their outlet sills were 10,108 120 and 779·834 mill. cft. respectively, or 92·84 and 7·16 per cent. of their total storage capacity. Deducting one tank, where the storage below outlet sill was proportionately very small, viz., 20 mill. cft. compared with that above it, 3,310 mill. cft., the storage capacity below outlet sill of the remaining 19 tanks was originally 10 per cent. of the total storage.

As the bottom contours are of small area, this allowance for silting will entail the raising of the outlet sill some feet above the lowest ground level of the dam. A further raising to any extent of the sill may not be desirable, if this will much lessen the storage to be drawn upon for a number of years, and increase its cost, before the silt occupies the space below the outlet, and will thus lead to a reduced area of irrigation and to a loss of return on the capital expenditure. Another consideration in regard to the level at which the sill should be placed, is that the higher such level, the greater will be the command per mile of its length of the canal led from the outlet. In some instances

this may justify the raising of the sill to a level higher than is necessary to allow space for the deposit of silt below it (para. 199, p. 284).

43. The Advantage of Earthen compared with Masonry Dams in respect to the Silt Difficulty.—Although it is economically not possible artificially to remove much of the silt deposited, the original storage, in the case of an earthen dam, can be easily restored when necessary, and at a cheap rate, by raising the full-supply level and the dam correspondingly. This is one of the chief advantages of an earthen dam over a masonry one, for in the case of the latter, increasing the levels is not so easy a matter unless that had been arranged for originally by providing a wider section to permit of it. If, however, this had been allowed for, it means that a considerable amount of the capital cost will be unproductive until the time for raising has arrived.[1]

VIII. Survey and Investigation Work.

43A. Irrigation Reconnaissance Survey.—Before a project is selected for detailed investigation, it is essential that the whole of the country in its neighbourhood should be examined by means of a reconnaissance. The principal objects of this preliminary survey are to obtain, at comparatively small expenditure of time and money, general information as to the nature of the tract examined, the facilities offered by it for irrigation, and the relative merits of all projects

[1] Since this was written, the Assuán Dam, Egypt, has been cheaply raised by a new method; in this the section is first widened by a strip of masonry on the downstream side, and then the top is heightened. "Minutes of Proceedings, Inst. C.E.," Vol. cxciv., Paper No. 4054. It was at first proposed similarly to raise the Bhátgarh Dam, near Poona, Bombay Presidency, but, subsequently, as the storage was to be very largely increased, an entirely new masonry dam of about 190 feet maximum height was constructed downstream of the original one which had a maximum height of 127 feet.

practicable in it. Such a reconnaissance enables a general plan to be drawn up so as to utilise those facilities in the best and most comprehensive manner possible, and so that each individual project proposed will work in, and will not clash, with other schemes practicable. Thus will be avoided the fatal mistake of carrying out irrigation work in a piecemeal way without reference to an all-inclusive plan.

To enable a proper comparison to be made of the relative advantages of competing schemes, approximate general surveys should be undertaken, and rough plans and estimates of the proposed works should be made. The designs for these should not be considered as final ones, nor the estimates for them as exact, but sufficient care should be taken in their preparation to obviate extensive alterations thereafter as these may greatly lessen the value of the preliminary work. In particular, that work should be based on cautious assumptions of the nature of the foundations, the character of the designs and the costs of the construction, as it is almost the invariable experience that the estimates of detailed projects amount to more than those of preliminary ones. Care should also be taken to prepare the designs and estimates of competing projects as far as possible on the same general lines so that a fair comparison as to their costs and advantages may be obtained.[1]

[1] The Máládevi site, dealt with in this book, (Plate 3), was selected many years ago after the valley of the Pravárá River, in which it is, had been thoroughly examined, and a better site could not then be found. Long afterwards in 1903 the Bhandardára site was discovered by the late Mr. Arthur Hill, C.I.E., and after some years was preferred. At it a masonry dam with a maximum height from foundation bed of 274 feet was constructed across a very narrow gorge —previously so high a dam was not contemplated. This dam was completed in 1926; it was then the highest in the world, but has since been slightly exceeded by the Exchequer Dam, Merced Valley, California. The Máládevi site, which is 11 miles to the east and lower down the river, is, however, not affected by this newer scheme, and can be utilised hereafter to provide additional storage when that is required. Objections have, however, been raised to it on account of the height of the earthen dam proposed and the nature of the foundations

44. Survey for the Works.—After the reconnaissance has determined which is the best scheme practicable, the first survey work to be undertaken in connection with the detailed examination of that project will be for the proper location of the dam line. A trial centre line should first be set out and contours should be run at convenient intervals, say 4 feet apart vertically, over the whole area which is likely to contain the final line. After this work is plotted to a large scale, the most economical line can be ascertained by trial in office, *i.e.*, by setting out on the plan different alternative lines practicable, and by estimating roughly the costs of dams to be constructed on them.

When the final line has thus been selected, it should be levelled over, points in the longitudinal section at every 100 feet being noted and also those where the ground slopes change. Longitudinal and cross-sections of the waste-weir and outlet channel lines should then be taken and the plan completed by running contour lines at 2 feet apart vertically over the whole area likely to be occupied by the works. From such a plan it will be easy to make any changes in the design which may be found necessary when working up the project (Plate 4, Fig. 1). The plan should be extended by ordinary survey so as to include the whole area on which temporary works may have to be constructed.

The areas from which the different soils required for construction are to be obtained should next be surveyed and plotted, and small trial pits excavated to test the depths of the soils. From this plan a tabulated statement of quantities, leads and lifts should be prepared in order to calculate the earthwork rate to be allowed for the construction of the dam. Similarly, the sites from which all other require-

ments, such as water, sand, lime, fuel, and building and pitching stone are to be procured, should be ascertained and the quantities available roughly determined so as to arrive at the probable cost of these materials.

45. Survey of the Capacity of the Reservoir Basin.—After this investigation has been completed, the reservoir basin should be carefully contoured. For large storages the contours are usually taken at vertical intervals of 10 feet apart, but for small ones cross-sections may be run at intervals across the basins, and the contours can then be plotted from them. It is as well first to fix the contour points by level and then to triangulate the areas comprised by them so as to ascertain the extent of those areas correctly, or, the survey can be carried out by the tacheometer. Where necessary, tie lines should be measured in addition. As village maps are not very accurate, the contour plan should be prepared independently of them, but the contours may also be laid down on these maps with reference to field boundaries, so that the levels of outlying temporary works may be ascertained at any time, and also, so that the extent of land to be acquired may be marked on them. The limits to be acquired should be defined permanently by stones, on some of which bench marks should be engraved. These stones should, as a rule, be fixed on field boundaries, and their distances from field marks should be measured, or off-setted, and recorded both on the plan and in a tabulated statement.

46. Rainfall and River Discharge Observations.—Rain- and river-gauges should be established as soon as the investigation of a project is commenced. In regard to rain-gauges it has to be remembered that a

single gauge may not be representative of the rainfall on the whole catchment, and will certainly not be so if it is of any extent, and if its physical character varies considerably (para. 14, p. 21). It is best, therefore, to fix a rain-gauge to ascertain the rainfall of each constituent area with a characteristic fall, and especially should this be done during the years in which the construction of the tank is in progress, as then there will be superior establishment available for checking purposes. With a series of such numerous observing stations continued for a number of years, it will be possible to ascertain the relative rainfall on the different constituent areas of the whole catchment, and when this has been found, the number of stations can subsequently be reduced greatly and the results from the diminished number applied to the whole catchment. There are several forms of automatic recording rain-gauges, but these are not often established in India, although in such cases they might well be fixed for checking purposes and to obtain records of the intensity of the rainfall.

The rainfall observations should always be supplemented by ones of river discharge, for these will give the actual amount of run-off and will thus take into account all the factors producing it. It is most important that the discharge available should be carefully ascertained, as on it depends the proper determination of the storage capacity of the reservoir. Similarly, high floods should be gauged so that the waste-weir may be correctly designed. The river-gauge should preferably be placed a little below the outfall of the waste-weir channel into the stream, so that observations of its discharge may be continued, if necessary, after the completion of the work. It

should be permanently set out and cross-sections of the gauge station taken, so that from them a tabular statement of the different discharges at different gauge readings may be made out and utilised in the future. For this purpose a reach of the river should be selected where the bed is uniform, both in longitudinal and cross-section, and where the velocity of flow is regular and not extremely great even during ordinary floods.

A clear overfall weir (paras. 161 and 176, pp. 212, 242), forms the most accurate gauging station; for important projects where such a weir is not available, one might be constructed, and might have the depth of flow over it continuously recorded by an automatic gauge. Where the expense of the construction of a new gauging weir will be considerable, a pair of automatic flood-level recorders might be established instead at some distance apart from each other, so that from them might be obtained a record of the varying surface slopes of the floods; thereafter, the discharge could be calculated from the velocities thus deduced and the areas of the flood sections.

IX. The Utility of Reservoirs.

47. The Utility of Reservoirs in Times of Scarcity and Deficiency of Rainfall.—There is an idea generally prevalent that, by the construction of numerous reservoirs, the country may eventually be protected from the worst effects of famine, but it is questionable if this is a correct one. It has been stated in paragraph 7, p. 11, that, in the case of storages situated in the plains and in the area of uncertain rainfall, the catchment areas, however large, in seasons of drought

may not be productive of enough run-off to fill them. If the run-off were then sufficient, this would imply that there would be a still larger quantity of rainfall in order to produce it, and this amount of rainfall would probably prevent entire scarcity. The real benefit of reservoirs constructed in such areas is that, when the rainfall is deficient on the whole, or is so irregularly distributed as not to be capable of bringing crops to maturity, their storages will be able to supplement it and to permit of the growth of some crops. There are many more years of deficient and irregular rainfall than ones of total scarcity, and it is during the former that reservoirs will be of substantial benefit and their construction will thus be justified.

While the failure of even the largest storages constructed in the plains may be expected in years of great deficiency of rainfall, the case is different with those situated in the ghâts, which have an unfailing rainfall. Although the rainfall there is also liable to great variations, still even in the worst years it is sufficient to cause run-off, which is all the greater owing to the steep, hard, and bare slopes which usually characterise the surface of catchments in these localities. It is therefore highly desirable that all such sites should fully be utilised first (as storages there are likely to produce most revenue and benefit to the people), and to postpone works in the plains until it is necessary to construct them for the purpose of employing relief labour.

48. **The Comparative Utility of Small and Large Reservoirs.**—A small tank, under the most favourable circumstances, will irrigate only a small area, and this, considered with reference to the whole country, will be of little benefit. Moreover, in times of scarcity of

rainfall, the replenishment of such tanks is likely to be deficient or totally to fail, and they will then be practically, or wholly, useless. The maintenance of tanks each having a storage of less than 50 million cubic feet, if attended to by Government, will be comparatively costly, and it will be more difficult to arrange for the supervision of numerous small works than of a few large ones having the same total storage.

The advantages of small tanks are that they will utilise small catchments which might otherwise be wasted, and will provide for the irrigation of isolated areas which might not otherwise be developed. If moreover, they are not too large, and each can serve only one or two villages, their management might be entrusted to the cultivators, Government attending only to the maintenance of the works.

Large reservoirs, on the other hand, will have larger and more unfailing catchments; will be capable of irrigating large areas which will sensibly affect the productive capacity of the country as a whole; their rate of storage will be less costly; and their maintenance will be comparatively cheap and their supervision easy. Owing to the variety of the interests they will have to serve, their management will have to be arranged for entirely by the Government.

In regard to mitigating the effects of scarcity the proper programme therefore seems to be to construct in the first instance large reservoirs with unfailing catchments, then those with less certain ones, and finally small tanks for the benefit of isolated areas.

49. Revenue Prospects of Reservoirs.—Owing to the low rates charged in India for irrigation, to the con-

siderable loss of water by evaporation, etc., in the storage reservoir, to the large quantity required to bring the crops to maturity, and to the great cost of storage works, it is doubtful if at first reservoir irrigation will ever be remunerative ; but, as time goes on and the value of the produce increases, the rates may be increased proportionately and the revenue results will consequently improve. Moreover, the benefit to the country as a whole has also to be taken into account: in normal years storage works will produce crops which will add to the wealth of the cultivating classes, while in an abnormal year works with an assured supply may easily mature crops the value of which will be equal to the capital expended on irrigation. Should an irrigation work pay a little more than its working expenses during normal periods, it may, from this point of view, be accepted as a scheme financially sound in regard to the country as a whole, although far from being a directly remunerative one to Government. If expenditure on relief during times of famine is devoted to the construction of such works, instead of being incurred on works of only temporary utility, each successive famine will thus permanently enrich the country and render it better able to withstand the effects of future scarcity.

It is useless to expect from irrigation from reservoirs the good returns which are derived from the large canals that are supplied by the great perennial rivers and do not require expensive storage works, as the natural conditions are unfavourable to the former and are favourable to the latter. The proper way of regarding reservoir schemes is to remember they are

the only ones available for the lands they serve, and that it is the duty of Government to develop the country so far as their finances will permit.

50. The Suitability of Reservoir Works for the Employment of Famine Labour.—The construction of reservoirs affords the most suitable work for famine labour. The quantity of work available on them for unskilled labour is so great that it will give employment for large numbers during the whole time of scarcity. It is thus unnecessary to incur the expense of moving the people frequently and of making numerous camps and other arrangements for them as occurs in the case of such less concentrated works as roads and railways. Moreover, the nature of the work is suitable for the agricultural population, that will comprise the large bulk of those who come for employment. The supervision of the people will be easier, and will be arranged for more cheaply, than it can be on any other less concentrated class of work.

An objection raised to reservoir works is that they cannot be completed in the one season during which alone it is considered likely that famine conditions will prevail. However, recent experience has shown that distress may last for more than a single season, and, even if not subsequently acute, it is fairly certain to exist in a more or less pronounced degree for some time during which the provision of employment is desirable in order to enable the people to recuperate both physically and financially. The construction of reservoirs will not interfere with, but will be in addition to, their usual employment, whereas, if in times of scarcity they are put on such works as metal-breaking

for roads, many will be deprived in succeeding years of their ordinary means of subsistence.

It is, of course, necessary that a well-considered programme of works should be arranged before a famine has to be dealt with, so that during it the construction of projects with poor prospects may be avoided.

CHAPTER II.

THE DAM EMBANKMENT.

I. Various Kinds of Dams.

51. Classification of Dams.—The different descriptions of dams may be classified thus :—

1. Purely earthen embankments ;
2. Earthen embankments with dry-stone toes (" compound dams," para. 128, p. 178) ;
3. Masonry dams ;
4. Composite dams ;
5. American types of dam.

52. Earthen Embankments.—A purely earthen embankment may be formed in one of the following ways, with :—

(*a*) A puddle wall at the centre, or, on the water slope, or, in some intermediate position ;

(*b*) An impervious hearting supported on each side by more stable material (this hearting is practically a very wide puddle wall formed of good soil only, not special clay carefully worked) ;

(*c*) A homogeneous section, without a puddle wall.

In English practice (*a*), having the central puddle wall, is the type generally adopted ; in recent practice in Bombay (*b*) has been followed ; while the earlier dams there were constructed according to (*c*), which, with the important modification of these examples, noted in paragraph 80, p. 118, is the type now recommended.

Earthen embankments with dry-stone toes (" compound dams ") may, in respect of the upper part, be

constructed in any of the ways adopted for purely earthen dams; they differ from them, in respect of the lower part, in having dry-stone toes to support the base when the work has to be carried to a great height.

53. **Advantages of Earthen Embankments.**—These two classes of earthen embankment are the ones described in detail in this chapter. The advantages they possess over other types are:—

(1) They do not require such expensive and solid foundations.

(2) The materials for their construction being necessarily close at hand (or they would not be economically practicable) labour on them can be concentrated and easily supervised, and they are therefore peculiarly well adapted for the employment of famine labour.

(3) They can be raised from time to time to meet the demand for more water, or to restore the deficiency of storage due to the silting up of the reservoir.

(4) They can be constructed quickly and by unskilled labour.

(5) They are the cheapest type, and, with suitable design and construction and by the adoption of proper precautions, can be made of any height which is ordinarily required in practice.

54. **Masonry Dams—Composite Dams.**—Masonry dams are the most stable of any form of dam. They, however, generally require deep and expensive foundations, and their construction is slow and costly. They cannot be raised beyond their originally designed height[1] unless their section has been made sufficiently

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[1] See footnote to paragraph 43, p 65.

wide at first for this purpose, a proceeding which involves the locking up of unproductive capital until the increase in height has been carried out. They are the most suitable for large and deep water-supply reservoirs, where risk of failure must be avoided at all costs, and are also the best for closing gorges with steep sides.

Composite dams are ones in which part of the length is formed as an earthen embankment and part as a masonry dam. In respect of costliness they are a mean between the two forms of which they consist. They are best adapted to sites where the river crossings are deep and the flanks are on high ridges. The junction between the masonry and the earthen embankment has to be most carefully made in order to prevent water finding its way through the structure at this point.

55. American Types of Dams.—Schuyler[1] gives the following classification of "rock-fill" dams, *i.e.*, those having an upstream :—

1. Facing of two or more thicknesses of planking ;
2. Facing of asphalt concrete laid on a sloping dry wall ;
3. Facing of Portland cement concrete laid on a dry wall ;
4. Facing of masonry built vertically and backed with earth which is covered on the downstream side with blocks of stone laid in mortar ;
5. Facing of steel plates laid on the upstream slope of a dry, hand-laid wall ;
6. Facing of earthen embankment ; or, having a
7. Central core of steel-plates without hand-laid facing walls.

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[1] Schuyler's "Reservoirs," p. 1.

Other types of American dams are :—

8. "Hydraulic-fill" dams;

9. Earthen embankments with masonry core walls;

10. Earthen embankments with steel-plate core walls.

When steel-plate cores are used, they are sometimes protected by casings of asphalt concrete, 4 inches thick, but these have been known to slip off [1] during construction, and the steel plates themselves to buckle [2] by expansion. In [1] one example the steel plate was $\frac{5}{16}$ inch thick for the lowest 20 feet, $\frac{1}{4}$ inch for the middle 20 feet, and $\frac{3}{16}$ inch for the top 28 feet.

Most of the facings described above appear to be of a temporary nature, and sufficient time has not elapsed since they were constructed to test their permanency thoroughly.

56. "Rock-fill" Dams.—The rock-fill dams are cheap to construct, but depend for safety on the downstream facing, for, if that is injured, or decays, the great pressure of water in the reservoir would rapidly disintegrate the hearting and carry away its constituent blocks, no matter how large. It is advisable to have fine material placed inside of and near to the downstream face so as to induce silting and staunching at the upstream face. In certain dams the upstream part is made of earthen embankment, and only the downstream part is of drystone construction.

There have been several instances of the failure of this class of dam, and the bursting of the Gohna Lake,[3] which was formed by a large landslip blocking

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[1] Schuyler's "Reservoirs,' p. 64

[2] *Ibid.*, p 22

[3] Selections from the "Records of the Government of India in the Public Works Department," No. cccxxiv.

the Biráhi-gangá, a tributary of the Alaknandá, (a principal affluent of the Ganges), in a narrow valley in the Himalayas, by a natural rock-fill dam of immense thickness (2,000 feet wide at the top, 11,000 feet at the base, and about 900 feet high), shows that this form of construction cannot be depended upon, so far as the hearting is concerned. It is true that the primary cause of the breaching of this mass was its being overtopped by the lake, but it early showed signs of failure due to percolation, which percolation eventually amounted to 350 cusecs.

57. "Hydraulic-fill" Dams.—The hydraulic-fill dam[1] is formed by sluicing an earthen area with a hydraulic jet and by conveying the resulting silt-charged stream on to the site of the dam, where the earth is deposited between shallow side banks, with the finer material as the hearting and the coarser as the facings, while the clear water is allowed to pass off. This form of construction requires :—

1. An abundance of water at the proper elevation to provide a sufficient "sluicing head";

2. An abundant deposit of suitable material for forming the dam, conveniently situated at each flank and high enough above the top of the dam to permit of the flow of the material required to complete it.

Where these conditions exist, the construction of the dam is both rapid and cheap, but in India the cheapness of labour, the scarcity of water and the expense of pumping will generally prevent the adoption of this form of dam making. This method of construction is considered in America to be a sound one; the objections to it are apparently that the layers may be

[1] Schuyler's "Reservoirs," pp 76–116.

deposited very wet (para. 117, p. 163), or may be stratified (para 118, p. 164), or may be consolidated solely by settlement (para. 119, p. 165). It would therefore seem proper to form the whole dam of self-draining material (para. 110, p. 155), to carry it up slowly so that it may have time to consolidate itself, and not to subject it to infiltration from the reservoir until settlement has practically ceased.

The failure of the important Necaxa Dam[1] in Mexico which was constructed by the old hydraulic-fill method is instructive. That work was designed to be about 1,200 feet long and of a maximum height of 190 feet: its top to be 54 feet wide and 16 feet above full-supply level: and its upstream and downstream slopes respectively to be 3 to 1 and 2 to 1. The central part was constructed of a pure clay of a highly retentive nature to a base width of about 365 feet and side slopes of about 1 to 1. This was to be kept in place by side fillings of material varying gradually from partly porous soil next the clay to rock-filling on the outer slopes; on the upstream side the side-filling was about 350 feet wide at the base and on the downstream side was about 250 feet wide. The total contents of the dam were estimated at 2,130,000 cubic yards, of which 1,926,000 cubic yards had been formed by May 20th, 1909, on which date a slip of 720,000 cubic yards of the upstream slope at the left flank occurred very suddenly, and the semi-liquid clay of the hearting flowed for 1,200 to 1,500 feet into the reservoir. The dam had then been raised to about 45 feet below its designed top. This slip compares unfavourably with the one of an ordinary dam shown in Fig. 17, p. 199.

[1] *Engineering News*, Vol lxii, No. 3, of 15th July, 1909, pp. 72–74, 77, and 78; *ibid.*, No. 4, of 22nd July, 1909, p. 99

It was expected that the side fillings would weight and drain the whole of the clay hearting but it was found that they had drained and consolidated the outer part only from 6 to 16 feet in thickness, and that the central part was so soft that six men were able to force 1-inch pipes into it to a depth of 50 or 60 feet. The downstream rock casing was of heavy limestone and had been carried up to full thickness and height; the upstream casing was of "tepetate" of only half the weight of the limestone, and, moreover, had been constructed of reduced thickness and height. This explains why the 2 to 1 downstream slope, usually the less stable of the two (para. 127, p. 178), was able to withstand the pressure of the soft clay hearting which carried away the 3 to 1 upstream slope. The dam was constructed very quickly and had continually on top a summit pond formed by the water from the hydraulic jets and this produced extra hydrostatic pressure.

The lessons in connection with hydraulic-fill dams thus constructed to be learnt from this failure are apparently:—

(*a*) A hearting of pure retentive clay and permeable side facings should not be formed, but the whole section should be of self-draining material;

(*b*) The work should be carried up slowly and evenly and to the full section, great care being taken to avoid stratification;

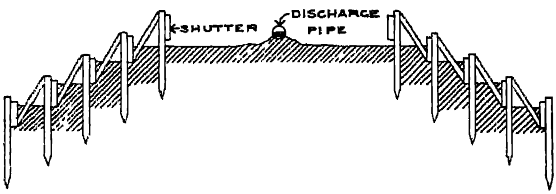
(*c*) The heaviest dry material available should be used for the outer casings and especially for the upstream one.

(*d*) As much time as possible should be allowed for the work to drain and consolidate before it is subjected to infiltration by water from the reservoir.

In modern practice[1] the defects mentioned above are avoided. The sluicing water (which is from eight to twenty times the volume of the dam) is not allowed to form a summit pool on the embankment but is run off at once. The material is deposited as a homogeneous mass without stratification by continually changing the dumping point, is made to flow longitudinally with a perfectly free discharge, and is confined laterally by rough wooden shutters supported by posts set out with reference to the correct side width and raised from time to time to keep pace with the embankment (Fig. C). Good proportions for the

FIG. C.

FORMING SLOPES—HYDRAULIC-FILL DAM



materials are one part by volume of clay to one of grit in the hearting, and two of grit in the outside casings.

58. Earthen Embankments with Masonry Core Walls.—American engineers are greatly in favour[2] of the construction of masonry core walls for earthen dams. One section[3] for the wall which has been

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[1] "Earth Dams and their Adjuncts," the Ambursen Company, 61 Broadway, New York.

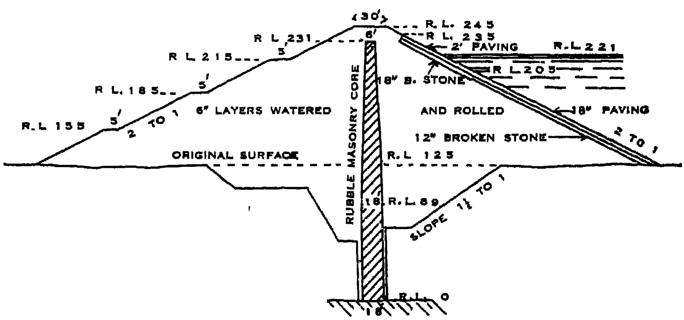
[2] Schuyler's "Reservoirs," p. 281. *Engineering News*, Vol. xlvii, p. 153, February 20th, 1902.

[3] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii., p. 245.

recommended is 4 feet or 5 feet thick at foundation level, 8 feet thick at ground surface level, and 4 feet thick at the top. Another was only 2 feet thick, uniformly from top to bottom. These sections appear too thin to cut off infiltering water under great pressure, and the first one, although superior to the second in this respect, appears to have been copied from puddle wall practice, and is not of a stable form for masonry. A much better section proposed for the new Croton Dam,[1] New York (Fig. 1), has the trench

FIG. 1

CORE WALL OF NEW CROTON DAM



portion 18 feet thick throughout, for 89 feet from foundation level to within 36 feet of ground level, and was thereafter battered uniformly on both sides for 142 feet up to its top, which was 6 feet wide and 14 feet below the top of the dam; that top was 245 feet above the foundation of the core wall. Even this thickness of core wall does not seem sufficient to stop entirely the filtration due to the enormous pressure, nor to be able to resist any unequal settle ment of the earth on the two sides, which is the great

[1] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii., p. 267.

danger to be feared in this form of construction. To prevent this unequal settlement occurring and producing lateral and unsupported pressure, the earthwork should be carried up uniformly on the two sides of the core wall and with its layers on each side sloping slightly to that wall.

The following advantages [1] are claimed for a masonry core wall:—

(1) If founded on an unyielding, impervious stratum it forms a perfect cut-off in the centre of the dam, preventing water which percolates into the upstream slope from reaching the downstream one. (Masonry, especially if thin, is not absolutely water-tight when subjected to great water pressure, but may be made more staunch by cement pointing, or plastering, the upstream face.)

(2) It cannot be washed out, as a puddle wall may be, should a leak through it be formed; in fact, such a leak through the masonry is more likely to be silted up than to be enlarged.

(3) It separates the dam into two distinct portions, an upstream one, which should be made as water-tight as possible, and a downstream one, which should be made as stable as possible. If these two kinds of earthwork abutted directly against each other without the interposition of the core wall, cracks might occur in the centre of the dam on account of differences in their settlement. (Such different settlements might still happen if the core wall were built and might then lead to its fracture. To obviate them the nature of the earthwork should gradually, not suddenly, be varied near the centre line of the dam.)

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[1] "Minutes of Proceedings, Inst C E.," Vol. cxxxii, pp. 267 and 268.

(4) It enables the outlet culvert to be carried through the dam with perfect safety.

(5) It allows the outlet tower to be replaced by a "dry well" tower, built upstream of, and in connection with, the core wall, thus dispensing with the need of an outlet tower and bridge (para. 207, p. 300).

(6) It gives an earthen dam greater strength to resist the erosive action of water passing over its top. (This topping of a dam should be prevented by providing sufficient waste-weir discharging power, and water should not be allowed to rise to within several feet of the top of the dam (para. 67, p. 95).

(7) It can be made to form a solid support to a crest wall protecting the top of the dam from wave-action (para. 70, p. 102).

A modified form of core wall has recently been devised. This is built hollow of reinforced concrete, with vertical cross-walls at intervals so as to make it cellular. The longitudinal walls are spaced sufficiently far apart to permit between them of inspection of the interior, and each is lined externally by a dry filtering layer, which collects the drainage of the earthwork of the dam and admits it through weep holes to the centre of the core wall, whence it is led out of the embankment by base drains on the downstream side. This design has been recommended for adoption in hydraulic-fill dams, as it is considered it will ensure the rapid drainage of the water-deposited material, and thus its early consolidation.

On the whole it appears that the masonry core wall is superior to the puddle wall, but its expense would probably be prohibitive in India.

To sum up, it may be said that in India rock-fill dams would be considered to be dangerous to construct in

view of their apparently temporary nature: hydraulic fill dams are not practicable on account of the absence of water at a high level; and earthen dams with masonry core walls are too expensive.

II. Theoretical Considerations.

59. The Deficiency of the Theory of Earthwork.—Properly designed and constructed earthen dams are amply sufficient to resist the pressure of the water which they hold up. The only way water can and does act prejudicially against them is by infiltration, which diminishes their frictional resistance and adhesion. The risk of failure lies in the liability of the earthwork itself to slip. There have been many mathematical investigations as to the behaviour of earthwork, but, naturally, these have been confined to laboratory experiments; and, although they are most useful in indicating the character of the forces at work, they cannot, from the nature of things, be based on actual and comprehensive data, and cannot, therefore, give the actual amounts of those forces in all the varying circumstances which occur in practice. Sir Benjamin Baker,[1] Past President Inst. C.E., has given numerous examples showing that the lateral pressure of earthwork against walls is, at most, only one-half of that pointed out by theory, and he states that practical considerations, rather than theoretical ones, should be taken into account when designing walls to resist earth pressure. Mr. (since Sir) G. H. Darwin,[2] M.A., F.R.S., concludes: "The soundest view seems to be that engineers have

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[1] "Minutes of Proceedings, Inst. C.E.," Vol. lxv., pp. 207 and 208.
[2] *Ibid.*, Vol. lxxi., p. 378.

no better practical course open to them than, neglecting the elaborate formulas which have been suggested, to work with semi-empirical rules such as those of Coulomb, and to allow a large coefficient of safety."

Rankine[1] has stated: "There is a mathematical theory of the combined action of friction and adhesion in earth; but for want of experimental data its practical utility is doubtful."

60. General Causes affecting the Stability of Earthwork.—Earthwork gives way by the slipping, or sliding, of its parts on each other. The resistance to this is due partly to the friction between the particles, and partly to their mutual adhesion or cohesion.

The friction[2] is measured by the angle of repose, and constants for it for different soils have been determined; these are co-efficients of the weight of the mass. Friction is greatest for coarse and least for fine soils; on it depends the permanent stability of natural and artificial earthwork. A slight addition of moisture increases the co-efficient of friction, but an excess of water acts as an unguent in diminishing that.

The adhesion, or cohesion, may be measured by the depth to which an unsupported face of earthwork will temporarily stand before that is affected by the weather: it gives additional stability to earthwork. It is an extremely varying force, depending largely upon the condition of the material. It is increased by a moderate amount of moisture, but is diminished by excessive wetness.

It is, therefore, evident that any given earthwork,

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[1] Rankine's "Civil Engineering," 11th edn., p. 324.
[2] *Ibid.*, pp. 315 and 316.

other things being equal, will be most stable when slightly damp, and least stable when charged with water. Hence its stability depends upon the ease and thoroughness with which it can be drained of superfluous and dangerous water. Professor Rankine [1] sums up the matter thus: "The properties of earth with respect to adhesion and friction are so variable that the engineer should never trust to tables or to information obtained from books to guide him in designing earthworks, when he has it in his own power to obtain the necessary data, either by observation of existing earthworks in the same stratum, or by experiment."

61. The "Historical Element" of Earthwork.—There is a further cause of variation in the behaviour of soils, and that is what Professor Clerk Maxwell has called the "historical element," which term not only comprises the manner in which the mass was put together, but also includes the different causes at work which have subsequently modified its condition.

In respect to the effect of the original method of the formation of earthwork on its stability and behaviour Mr. (since Sir) G. H. Darwin,[2] M.A., F.R.S., made experiments on "The Horizontal Thrust of a Mass of Sand," which show that when sand is formed in different ways into an embankment it produces different amounts of thrust, although in each case the mass presents the same external appearance. This fact is recognised in engineering practice, for, when it is desired that earthwork should settle tightly against a retaining wall, its constituent layers are

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[1] Rankine's "Civil Engineering," 11th edn., p. 317.
[2] "Minutes of Proceedings, Inst C E," Vol. lxxi

made to slope downwards towards the wall; the steeper the slope, the greater will be their pressure. If, however, the wall is to be freed from earth pressure, the layers are made to slope downwards away from it. The same thing occurs in Nature. "The stability of sedimentary rocks in the side of a cutting is greater when the beds are horizontal, or dip away from the cutting, than when they dip towards it."[1]

Recent experiments[2] on earth pressures showed that the amount of penetration of a weighted plunger into sand, sifted garden earth, and sifted ashes and cinders (dry, homogeneous materials) varied fairly regularly with the pressure: while that into clay increased enormously with the pressure, indicating that for it the internal co-efficient of friction falls off rapidly with increase of pressure. These experiments thus confirm the remarks made at the end of the next paragraph as to the necessity for taking special precautions in the design and construction of dams of considerable height. The angle of internal friction is not constant for any material, but varies with the degree of consolidation of its particles, and is the same as the angle of repose only when the material which is tested for penetration is in its naturally loose state, in which it is alone possible to measure its angle of repose. The angle of repose thus gives the worst condition of stability, and if adopted as done by Rankine for determining the amount of earth pressure against a retaining wall, provides an ample factor of safety for ordinary working conditions: it, of course, varies with the nature of the material employed.

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[1] Rankine's "Civil Engineering," 11th edn., p. 318.

[2] "Experiments on 'Earth Pressure,'" by P. M. Crosthwaite, B.A.I., M.Inst.C.E., "Minutes of Proceedings, Inst C E," Vol. cciii., Paper No. 4194.

62. The Effect on Earthwork of Causes acting subsequently to its Construction.—Attention has not always been paid to the modification of the behaviour of earthwork by the effect of causes acting on it subsequently to its construction, although it is equally necessary to take this change into account. In nearly all earthworks the practice is to treat the material as homogeneous from top to base, and to adopt a uniform slope throughout. The lower portions in a large dam must, however, be in a very different condition to that of the upper ones, as they are much more highly compressed and are moister. Probably the enormous superincumbent weight causes some stratification of the lower parts and diminishes their cohesion, while the increased smoothness, due to the pressure, lessens their frictional resistance. The amount of increase of frictional stress, according to the depth below the surface, depends upon the viscosity of the earth enabling it to transmit pressure, and this pressure must vary from point to point on the cross-section of the dam. The increase in moisture at the base will diminish both the frictional resistance and the cohesion. The variation of the materials, their disposition and the methods of construction, introduce further elements of change, so that there are numerous, entirely hidden forces at work the magnitude and resultant action of which can be determined only from the experience of the works themselves.

In Nature it is seen that hills, or even large masses of soil, have not an even slope, but one that varies from steepness at the top to flatness at the base (*e.g.*, Fujiyama, in Japan). Although this form is partly due to the effects of denudation, it is also partly due to the natural slopes assumed by the

material of the hills. Slips of earthwork show, at first, a similar, but more pronounced, outline. It is known, moreover, that the limit of height of ordinary earthen dams is comparatively low. French engineers have placed it at 60 feet, and, although there are many instances of greater heights having been successfully accomplished, a considerable amount of care is necessary to ensure the safe construction of dams of a height greater than 50 feet (paras. 66, p. 95, and 127, p. 177). Low dams can be constructed with much steeper slopes than high ones; the water-faces of dams require a flatter slope than the rear ones. From these considerations may be deduced that, in an originally homogeneous high earthen dam with plane slopes, the resistance to slipping decreases with the height from the top, and that the proper section for it is one having the slopes continuously flattened towards the base. An "empirical section" based on these principles is shown in dotted lines on Plate 5, Fig. 2.[1] Taking the whole cross-section into account, it will be seen from this that a very considerable flattening of the base slopes results in a comparatively small increase of the original area of the cross-section.

63. The Behaviour of Puddling Clay with Water.—Experiment [2] has shown that a good natural specimen of clay when dried lost 25 per cent. of its weight, and 10 per cent. of its bulk; it then became extremely compact, and, if not allowed to expand, offered great resistance to the passage of water. A dried specimen of this clay reduced to a fine powder absorbed about

[1] Mr. A. R. Pollard, B A., M.Inst.C.E., in paper No. 4603 contributed in 1927 to the Inst.C.E, discussing a mathematical formula for the profile of earth dams, says. "The value of the constant $c = 0{\cdot}055$ gives a natural curve of repose that is almost identical with the upstream 'empirical profile for earthen dam' . . ." (given in Plate 5, Fig. 2).

[2] *The Builder*, Vol. li., p. 400

75 per cent. of its weight of water, and, when not confined, allowed of free percolation. When this powdered clay was pressed into a tube, 8 feet long and 3 inches in diameter, it absorbed 35 per cent. of its weight of water, but there were no traces of filtration through the tube. The compressed particles of clay, in absorbing the water, expanded so as to become watertight; the greater the pressure of the water, the more satisfactory were the results.

On a large work it would not be economically practicable thus thoroughly to desiccate clay, nor feasible to consolidate the whole mass if dry. The experiments show, however, the advisability of using no more water in the construction of earthwork than is necessary to produce compactness by rolling. The further compression of the mass will result from the superincumbent weight of the dam, which should be allowed to act for as long a time as possible before the reservoir commences to fill, so as to prevent filtration through green material. In a wet state clay reaches its extreme limit of expansion, and, when then exposed to the action of water, filtration is likely to take place between the separated particles. Clay is, moreover, so retentive of water, that, if once soaked, it will be long before it parts with the excess of moisture; hence the greatest care should be taken during construction to use the minimum amount of water.

64. The Rate of Filtration through Soils—The Depth of the Puddle Trench.—The rate of filtration through a soil depends upon its porosity, which governs the frictional resistance to flow, the slope and length of the filamentary channels along which the water may be considered to pass and the pressure head on it. It is evident, therefore, that the direct rate of infiltration

in a homogeneous soil must decrease from the top to the bottom of a puddle trench. The best section for a puddle trench is thus a truncated wedge, such as an open excavation would give. It is true that the uppermost infiltering filaments, when stopped by the puddle, will endeavour to get under it, but a depth will eventually be reached when the frictional resistance along the natural passages will be greater than that due to the transverse passage of the puddle trench, and it is when this occurs that the latter may be stopped without danger, as the filtration to it will be less than that through it. This depth requires to be determined in each case, but in fairly compact Indian soils 30 feet will be a fair limit for a reservoir 60 feet deep (para. 86, p. 124). Mr. David Gravell[1] cites the opinion of Sir Robert Rawlinson, that 30 feet depth of puddle trench is sufficient if a thick bed of concrete is placed at the back with a well for collecting water and a pipe leading this off to the downstream side; this was done in the case of the Yarrow Dam.

III. The Design of Dam Embankments.

65. The Section of the Dam Embankment.—The proper section to be adopted for a dam embankment depends upon:—

1. The angles of repose of the soil of which it is formed, when dry and when saturated by the water of the reservoir or by rainfall;
2. The nature of its material;
3. The nature of the foundation;
4. The height to which the work has to be raised;
5. The importance of the work.

[1] *The Engineer*, Vol lxiii., p. 189.

The following table gives the general sections which may be adopted with safety and economy for all ordinary good soils properly consolidated and resting on good foundations :—

1	2	3	4	5	6
Height of Dam above Foundation Level	Height of Top of Dam above H F L	Top Width	Upstream Slope.	Down-stream. Slope	Width of Dam at H F L
	Feet	Feet	Ratio of Horizontal Width to Vertical Height		Feet.
1. 15 feet and under	4–5	6	2 to 1	$1\frac{1}{2}$ to 1	20–$23\frac{1}{2}$
2 15 feet to 25 feet	5–6	6	$2\frac{1}{2}$ to 1	$1\frac{3}{4}$ to 1	$27\frac{1}{4}$–$31\frac{1}{2}$
3. 25 feet to 50 feet	6	8	3 to 1	2 to 1	38
4. 50 feet to 75 feet	7	10	3 to 1	2 to 1	45

Above 75 feet in height special precautions have to be taken : these are described in paragraph 128, p. 179.

In any dam it is advisable to preserve much the same section throughout so as to improve the appearance of the work as a whole, and, also, because reductions at the flanks do not effect much saving unless these are very long and moderately high. In a long dam three changes at most will suffice, different sections being adopted for :—

1. The Gorge Embankment ;
2. The high part of the Embankment ;
3. The low Flank Embankment.

To facilitate the work of setting out, when such changes of section are made, they should be carried out in lengths of 100 or 200 feet, instead of uniformly throughout the length of the dam.

66. The Height of the Dam.—The height to which a dam has to be constructed depends upon :—

1. The amount of foundation clearance ;

2. The ground levels ;

3. The full-supply and high-flood levels ;

4. The amount of "free board," or the height of the top of the dam above high-flood level ;

5. The importance of the particular section of the dam, considered with reference to that of other parts of the embankment.

6. The allowance for settlement.

(1) Is dealt with in paragraphs 111, 113, 114, and 115, pp. 157–163 ;

(2) depend upon the longitudinal section of the dam line ;

(3) the full-supply level is determined by the amount of storage to be impounded, and the high-flood level, by the discharging capacity of the waste-weir.

(4) and (5) are dealt with in paragraph 67, below, and (6) in paragraph 72, p. 105.

French engineers consider that the maximum safe height for an earthen dam is 60 feet, but embankments have been constructed in the Bombay Presidency up to a height of 80 feet. In England there are several instances of dams having been raised to 80 feet,[1] some to 100 feet,[2] and one to 125 feet [3] (see also para. 127, p. 177).

67. The "Free-board" of the Dam.—The dam has to be raised a certain height above the high-flood level as a matter of safety in ordinary circumstances, and to provide for any accidental settlement which may occur. This height depends not only upon the size of the reservoir, but also upon the importance of the particular portion of the dam considered with reference

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[1] "Minutes of Proceedings, Inst, C.E.," Vol. clxxxii, p. 250.
[2] *Ibid.*, p. 208.
[3] *Ibid.*, p. 205.

to that of other parts of the embankment. For large reservoirs, as the necessity of their safety is greater, the free board should be larger than it is for small ones ; hence it is advisable to increase the freeboard of the dams of the former by at least a foot more than that sufficient for the latter. Similarly, where the dam is of extreme height, as where it crosses the bed of the impounded stream, it should be raised at least a foot higher than at the lower flanks, so that the danger may be avoided of a breach occurring from any cause at the former place, before doing so at the latter. At a low flank a breach will usually cause a smaller and less damaging flood than that which would occur were it to take place at the gorge embankment ; moreover, the subsequent repair work at the former would be relatively much cheaper to effect and would be safer afterwards than it would be at the latter.

The free board does not depend directly upon the full-supply of the reservoir, as the required margin for safety has to be given above its high-flood level.

68. The Top-width of the Dam.—The top-width of the dam should vary according to the size of the reservoir and to the height of the dam, so as to be in accord with the general scale of the work, and, principally, to allow of the top-level of the dam being restored should any extra settlement occur. As a decrease of top-width affects only slightly the total area of the section of the dam (para. 73, p. 107), it is, as a rule, best to maintain the top-width the same throughout the embankment, and, if any reduction is contemplated, to confine it to the low ends of the flank embankment. Where such changes of top-width are decided upon they should be carried out in lengths of 100 or 200 feet, so as to facilitate setting

out, and should not be continued uniformly throughout the whole length of the embankment.

The top-width of a dam forms a footway along the work. It is not, however, desirable, either in the interest of the users nor of the proper maintenance of the embankment, to carry a roadway along the crest. When a roadway is necessary, it should be formed on top of a berm and near ground level.

69. The Side-slopes of the Dam.—These primarily depend upon the nature of the material of which the dam consists. In paragraph 80, p. 118, it is stated that it is advisable to make this material of the same description in all cases, so that the same side-slopes should, everywhere, meet the conditions in force. Moreover, the side-slopes given in the table in paragraph 65, p. 94, have been found in practice, in numerous examples of different kinds of earthwork, to be reliable, as they are sufficiently flat to resist slipping.

The upstream slope is made flatter than the downstream one as it usually consists of more clayey materials and is saturated by the water of the reservoir.

When the embankment is higher than 75 feet, it will be necessary either to change these slopes or to make a difference in the design of the dam.

As shown in the table in paragraph 65, p. 94, the lower the dam, the steeper may be its side-slopes; it is thus permissible slightly to steepen the side-slopes near the top of a dam (Plate 5, Fig. 2), but not generally desirable, as thus raising after settlement becomes more difficult.

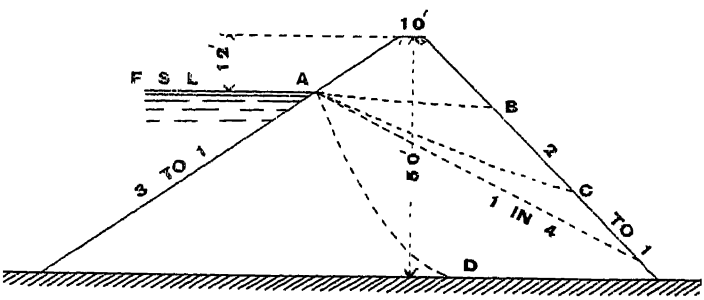
69A. The Effect of Percolation through the Dam.—The side-slopes should depend not only upon the material of the dam but also upon the way in which

it was laid and consolidated, as infiltration from the reservoir will decrease the weight of the submerged part of the embankment by that of the water displaced by it, and will thus diminish the stability of the dam in proportion to its porosity. For the same reason, the heavier is the material of which the dam is constructed, other things being equal, the more stable will be the structure (para. 128, p. 179).

The effect of percolation through a dam is illustrated in Fig. 1A, which shows several percolation lines. These lines indicate the various hydraulic gradients

FIG. 1A

HOR^{L} TO VERT^{L} 1 TO 2



of internal flow through the dam and the corresponding levels to which the saturation water will rise in it. The slope of a percolation line measures the resistance of the material of the dam to the flow of water through it, and the decrease in weight and stability of the dam is measured by the height at any point of that line above the ground line.[1] In addition (para. 60, p. 87), the saturated portion has

[1] "Public Water Supplies," by Turneaure and Russell, 1st edn., 1907, pp. 323–325.

diminished frictional and cohesive resistance to slipping.

If the dam is made of extremely porous material offering hardly any resistance to infiltration, the surface of the water percolating through it will be nearly level, such as is shown by the line AB. If the material is more compact but still somewhat porous, that surface will assume a line such as AC. If, however, the dam is made of thoroughly consolidated watertight material, the internal percolation line will be somewhat as shown by AD.

The first case, AB, could occur only in a bank of dry rubble, etc., the particles of which were separated by wide interstices; those particles being of large size, the bank would be stable until the velocity of the water between them increased sufficiently to carry them away (para. 56, p. 78). The second case, AC, might happen in a badly constructed earthen dam, and the amount of infiltration shown would probably cause a slip. The third case, AD, represents what occurs in a properly consolidated dam, more especially if its down stream portion is formed of self-draining material (para. 110, p. 156), and is underlain by base drains (para. 112, p. 159) so as to secure a dry and thus a thoroughly stable downstream toe.

This illustration indicates the importance of constructing the upstream part of the section with impervious material, and of increasing its resistance to infiltration by thorough consolidation. Every precaution should be taken to resist percolation on the upstream side of the centre line, and to drain off harmlessly on its downstream side the small amount of water which reaches the centre line so that there may be a substantial cover of dry material

on that side above the line of saturation of the dam.

Experiments to test the surface line of percolation have been made on certain dams in Bombay: these have shown in bad examples that the line slopes about 1 in 4 from the reservoir surface, but in good ones is somewhat steeper. The experiments were made by sinking small iron pipes with perforated ends from $1\frac{1}{2}$ to 2 inches in diameter through the embankments; they probably exaggerate the actual state of affairs, as doubtless the water imprisoned in the dams heads up in the pipes (see Elkington's system of subsoil drainage described in para. 108, p. 153). Anyhow, slips in the dams are there of rare occurrence although some of the experiments would indicate they should take place as the percolation lines determined have a tendency to meet the downstream slope of the dams above their bases.

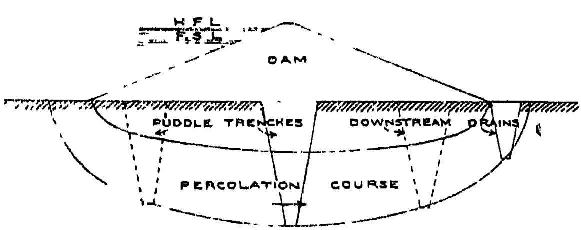
If a small amount of percolation took place uniformly throughout a solidly founded dam, and did not carry away any of its material, it would not seriously affect its stability, as is proved by many existing dams, of which few are quite impervious, and in cuttings through leaking strata. The danger is that the percolation water may sodden the base, or may be concentrated from a length of the dam and endeavour to find a defined outlet, such as a settlement crack or pervious layer. If successful in this, the subsoil flow may be able to detach from the main part of the embankment the portion of the earthwork thus separated and cause it to slip. As long as the percolation water issues clear and does not increase in amount, there is no fear that an accident will be caused by it. Percolation can be reduced by careful selection and

proper consolidation of the material of the dam: such that still takes place should be dealt with by drainage.

69B. Percolation below the Dam.—Percolation below a dam depends upon the location of the subsoils of varying porosity and usually will take place through the most porous layers even although they are deep-seated. It therefore becomes highly important to cut off the flow of such and to ascertain that they do not exist below the foundation of the puddle trench which is designed to secure the staunchness of the dam. Theoretically the lines of flow of subsoil percolation below a dam on homogeneous

FIG. 1B

PERCOLATION BELOW DAM



foundations and having equal side slopes are a series of confocal ellipses[1] (Fig. 1B): this is owing to the effect of the weight of the dam, which increases from its toes to its centre line, and to the resistance offered by the central puddle trench. The rate of flow is greater the shorter the line of flow, and is thus greatest at the exit near the downstream toe. From the Figure it will be seen that the central puddle trench has to be of the maximum depth, and that the trench

[1] Parker's "Control of Water," 2nd edn., p. 293. George Routledge and Sons, 1925.

could be reduced in depth without diminishing its efficiency by placing it more upstream; there are, however, practical objections to this (para. 85, p. 122). It will also be noticed that a drain parallel to the dam and just outside its downstream toe will tap the subsoil flow passing the puddle trench as effectively as a deeper drain formed under the dam itself. The former position is therefore best for the downstream drain mentioned in paragraph 107 (*b*), p. 149; moreover it is not advisable to have a deep drain under the dam, as there it cannot easily be attended to subsequently should that prove necessary (Fig. 11, p. 151).

70. The Crest of the Dam—the Crest Wall.—In the earlier Bombay examples both the side-slopes above high-flood level were reduced to 1½ to 1 for reasons of economy. That practice is now condemned, as it does not allow any margin for making up any excess settlement that may occur.

There is another way of finishing off the dam, and that is by a crest wall as illustrated in Plate 5, Fig. 1. This has a section with faces battering to the centre line of the dam both on the upstream and downstream sides, so that the wall cannot separate from the embankment, and so that it has ample strength with which to resist the thrust of the earthwork. To prevent any settlement from occurring, such a wall should have a wide concrete foundation, and should not be built until the dam has obtained a practically final consolidation. With proper arrangements such consolidation can always be attained, although, of course, the delay involved by them is not a recommendation in favour of this form of construction. The toe of the wall should be protected by a strong

apron, 2 feet thick, of close-fitting pitching to prevent it from being undermined.

The advantages of this crest wall over the ordinary slope are that it will:—

1. Protect the dam up to its extreme top from wave-wash, the inroads of vermin, and the growth of thick vegetation;
2. Act as a wave breaker and prevent waves from being carried over the dam in severe storms;
3. Lighten the top of the dam and save the construction of a large amount of earthwork which would be entailed by extending the ordinary dam slope to the top of the dam;
4. Permit of the raising of the dam for a moderate height when settlement occurs without a great reduction of its top-width, the crest wall being then continued with its original or reduced batters;
5. Give the work a better finish;
6. Effect, at ordinary rates, some economy, compared with the ordinary continuous 3 to 1 slope, when the dam is over 27 feet in height. If the top of the dam is widened, as well it may be with this design, this saving will take place when the dam is correspondingly more than 27 feet high.

Crest walls have, so far as is known, not been constructed in India on dams of any height, although there are many examples of old native works of small height which have been formed with an upstream wall backed by embankment. In England crest walls have been built, but possibly to a different section, and with fewer precautions having been taken. The objections there raised to them are:—

1. Waves[1] wash up the pitched slope, strike the wall, and, rebounding, undermine its base;

2. Waves[2] are apt to be carried over the wall in stormy weather and thus to erode the earth backing.

These objections are against the experience of harbour practice in which vertical walls are now preferred to slopes, as they are found to lessen wave action. The replacement of the top-slope of the dam by a crest wall is, however, not a matter of great importance, and the general practice hitherto has been against it.

71. The Width of the Dam at High-flood Level and Full-supply Level.—The width of the dam at high-flood level depends upon the height of the top of the dam above it (taking the ordinary free board and the allowance for extra settlement into account), the side-slopes and the top-width. The figures in the table in paragraph 65, p. 94, show that this width is a considerable one, and is unnecessarily large if resistance to water infiltration had alone to determine it, for, in that case, a width of 6 feet would be ample.

The width at full-supply level cannot be given in that table, as it is equal to the width at high-flood level *plus* the sum of the ratio of the slopes multiplied by the high-flood depth, which depends upon the discharging power of the waste-weir and will thus vary for different reservoirs. This width is also far in excess of that which is required only to resist infiltration, and, generally, the widths at each level of the cross-section are, similarly, greatly in excess of those necessary for this purpose when the dam is well constructed; they are, in fact, determined by wholly different

[1] "Minutes of Proceedings, Inst C E," Vol. cxxxii, p. 205.
[2] *Ibid.*, p. 208.

considerations, viz., the height, the side-slopes, and the top-width required for the dam as a whole.

72. Allowance for Settlement.—No matter how well a dam has been consolidated during its construction, its enormous weight, which much exceeds any that can artificially be brought to bear upon it, will further compress the earthwork. The result of this amount of further compression is known as settlement, and provision for it must be made, both in setting out the work and in estimating its quantity. The amount of vertical settlement of a well-consolidated dam should not exceed $\frac{1}{30}$ to $\frac{1}{36}$th of its total height, measured from its cleared foundation to its designed top.

During the monsoon the moistness of the air and the rainfall have great effect upon the settlement of a new dam, and it is probable that during the course of the first monsoon an embankment, although not called upon to impound water, will attain half its final amount of settlement. If, owing to the height of the section, the dam has to be completed in more than one working season, it will suffice, when setting out the work for the second and subsequent years, to consider that the finished base has already attained half its final settlement and to adjust the setting out of the upper part accordingly.

The practically complete settlement of the dam will be attained in a few months after the full depth of storage comes against it. The dam will continue to settle for a few years more, but only to a very small extent, and, five years after water is admitted against it, there should not be any sensible further settlement.

It will not, however, be right entirely to depend upon

this property of the self-consolidation of a dam and wholly to neglect artificial consolidation. The latter enables the whole mass to settle uniformly and gradually, whereas, if the earthwork during construction were simply deposited in place, a much larger amount of settlement would rapidly occur as soon as the dam became wetted, and, as this could not be uniform, owing to the varying heights in the longitudinal and cross-sections of the work, internal stresses would be set up. Moreover, were the embankment composed originally of loose material, the water of the reservoir would find its way into it for a considerable distance and would tend still further to produce unequal settlement and greater internal stresses, even if it did not cause the dam to leak or burst.

The more a dam is consolidated artificially, the less will be its subsequent settlement and the freer will it be from internal stresses, as its consolidation will be more uniform. To ensure uniformity and reduction of settlement it is desirable to construct a dam slowly, so as to let its power of self-consolidation come gradually into play.

73. The Sectional Area of a Dam Embankment.—The sectional area of a dam with uniform side-slopes is given by the formula:—

$$A = \left\{ T + H\left(\frac{S_1 + S_2}{2}\right) \right\} H$$

where A = the area in square feet;
T = the top width in feet;
H = the total height in feet; and
S_1 and S_2 = the ratios of the horizontal widths of the two side slopes to their vertical height.

It will be seen from this that in high dams the top-width is a comparatively small factor and that the areas of sections of different heights practically vary as the squares of their heights multiplied by half the sum of the ratios of their side-slopes.

In Appendix 20, p. 440, are given tables of sectional areas for all ordinary top-widths, side-slopes, and heights varying by 0·1 of a foot; further refinement in taking out the height is unnecessary. It must be remembered that H is the sum of the natural height, the allowance for settlement (para. 72, p. 105), and the allowance for foundation clearance (para. 113, p.160).

74. Sections required at Particular Sites.

(*a*) *Gorge Embankments.*—Where the gorge, or river crossing, is very high, a particular treatment is necessary: this is described in paragraph 128, p. 178, which deals with dams having drystone toes. Where it is of considerable height, the dam will, as explained in paragraphs 67, 68, and 69, pp. 95–97, require a greater free-board, greater top-width, and possibly flatter slopes than when it is of small height. At a gorge particular attention should be paid to the benching of the side-slopes of the ground, as they will otherwise tend to cause the embankment to slip off them. To prevent this the benching should be designed with base-slopes inclined downwards from the natural gorge-slopes, so that the earthwork of the dam during settlement will tend to move towards the flanks rather than towards the river crossing.

Some English engineers prefer to slope off the sides of the gorge to smooth surfaces with the object in view of making the gorge embankment wedge-shaped in longitudinal section, and thus to ensure during settlement that the earthwork will be forced tightly on to

those sides. However, on account of the greatly varying heights of the embankment at, and near the sides of the gorge, the settlement of the gorge embankment will tend to make its earthwork leave that of the flank embankments; it therefore seems better to counteract this tendency to separation by benching as described above. Another important consideration is that if the settlement of a high mass of earthwork is facilitated, too rapid motion may be caused and may result in a slip: to prevent this from occurring it appears advisable to retard the motion during settlement by benching the sides of a gorge. For low masses of earthwork, where slips cannot be induced by settlement, it is undoubtedly best to adopt the smooth wedge-shaped section (end of para. 91, p. 129).

Where the slopes are precipitous, it is better to substitute a masonry dam for an embankment, so as to avoid the tendency to slipping of the earthwork at such a place. The former design will have the added advantage that under-sluices can be made in it which will aid the waste-weir by bringing early and safely into action the "flood-absorptive capacity" of the reservoir (para. 184, p. 253).

(*b*) *Dams on Inferior Foundations.*—A really bad foundation should, of course, be avoided, as no treatment, short of removing all the defective material, will enable the dam to be constructed safely at the site. There are, however, other foundations which are not good, but which can be utilised by special arrangements. The principal of this class is deep black "cotton-soil," which so often exists at the sites of dams. Where this occurs, the side-slopes of the dam should be widened at the base, so as to distribute the

weight over as large an area as possible, and, as far as safety permits, should be steepened at the top of the section so as to reduce the total weight. Particular care should be paid to drainage, and, a short distance inside the downstream toe of the dam, should be a deep and wide trench filled with pervious, sound, and stable material to prevent the movement of the subsoil. The base of the downstream toe should be founded on a layer of muramey, or shaly, material, say 5 feet thick near the centre line of the dam and gradually reduced say, to 3 feet at the downstream toe (see also para. 114, p. 162).

75. Breaching Sections.—In paragraphs 66 (5) and 67, pp. 95, 96, it has been explained that the more important parts of the dam should be raised higher than the less important ones, but it is desirable that even the latter should not be injured in abnormal circumstances. To guard against such damage "breaching sections" should be introduced wherever safely and economically practicable. These sections should, as a rule, have a height about 2 feet less than that of the flank embankment, their top-width should be reduced to 6 feet, and their side-slopes steepened as much as is quite safe in ordinary conditions. They should preferably be located where there are natural saddles in the ridge line, so that any flood resulting from their being breached may be confined by the rising ground on each side. Where such a saddle does not exist, an artificial one can be formed by excavating, downstream of the breaching section, a channel to lead the flood safely away from the main dam. The excavated spoil can generally be used for the construction of the dam without increased cost.

To be effective the breaching section should be long

if the embankment at it is low; if that is high, its length may naturally be shorter. Too great a height of dam is, however, disadvantageous, for if the breaching section were formed there, it would continue longer in flow with a larger discharge, that would tend to produce a deep scour channel which might lower the reservoir unduly and increase the cost of reconstruction. As a general rule, it may be said that the breaching section should be situated where fairly hard material is not more than 4 feet below full-supply level. Should such material not exist, it may be necessary to construct a curtain wall across the breaching channel close to the toe of the dam so as to prevent excessive scour of its base.

The best position for a breaching section is where the channel from it can be led into the waste-weir tail channel, as then the flood from it will do the minimum amount of damage. For this reason, when the waste-weir is situated at an independent saddle, its flank embankments can be made to serve as breaching sections (Plate 6, Fig. 1, and paras. 162, p. 215, and 191, p. 266).

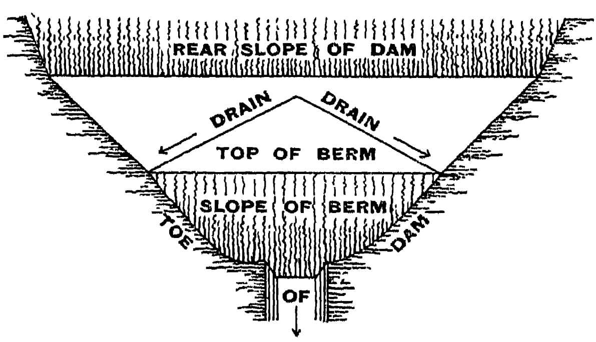
It is not absolutely necessary that the breaching section should breach automatically; it will suffice if its section is reduced in area so that it can quickly be cut away when necessary, for, owing to the slowness with which a reservoir rises, even during a flood, there will generally be ample time for this action to be taken so as to save the main dam from being destroyed.

76. Berms.—In some large works, instead of flattening the downstream slope to get an increased base-width, the increase has been obtained by adding a berm, having the same, or nearly the same, side-slope as that of the upper part of the dam. Assuming

the theory of the angle of rest of earthwork to be correct, the whole section is not in so stable a condition as it would be if the material of the berm were distributed throughout the dam so as to flatten its slope. The frictional resistance to slipping of a dam is a measure of its weight, and is independent of the area of its base, but its cohesive resistance is a measure of that area; the construction of a berm increases that area more than the material in it would do were it distributed all down the slope, so that, in this

FIG. 2

PLAN



respect, a berm is useful. The chief advantages of a berm are that, by its sudden increase of the section of the dam, it tends to prevent any dislocation at the top from extending to the base and any bulging of the subsoil at the toe: further, it virtually reduces the height of the ordinary section of the dam where that is suddenly increased, as at the river crossing. Finally, in the case of a badly constructed dam with a flat hydraulic gradient (Fig. 1A and paras. 69A and

151, p. 99, 202), the addition of a berm will weight and buttress the toe of the embankment by affording a cover of dry material to the unduly saturated base.

A berm is also useful for carrying a road across the river gorge and for passing off from the surface of the dam the drainage due to heavy rainfall, which amounts to a considerable quantity flowing at a great velocity when the dam is a high one, and, if not diverted from the base of the embankment, might cause the formation of scour channels down the slope. Fig. 2 shows how this can be done by means of paved, water-tight drains leading the drainage of the dam to the natural ground on its flanks. Such drains, if not water-tight, would be sources of danger, as they would tend to produce lines of supersaturation in the heart of the earthwork which might lead to the formation of slips. For this reason it is not advisable to form diagonal drains down the slope of the dam to pass off its drainage; in one instance where these were laid, slips were thus caused.

An objection [1] raised to a berm or "hump" is that it may tend to overweight the slope of the dam below it, and make it subside from the hearting, thus causing a fissure. Such action can take place only if a dam is made of non- or loosely-compacted material. When a berm is part of the original design, it will be formed and consolidated simultaneously with the main dam and will not separate from that. If subsequently added, it should be made with layers slightly tilted downwards to the dam so as to prevent such separation. If a temporary works road is added to the dam it may have the above effect. It should therefore be formed with the dam as that is raised.

[1] Parker's "Control of Water," 2nd edn., p. 310.

Berms[1] were used in early English practice, but English engineers now appear to prefer gradually to flatten the slope of the dam than to step it out in a series of berms on which water might lodge. To prevent such lodgment of water, the top of a berm should have a slope of about 1 in 20 downstream.

It is not recommended that there should be a series of berms up the slope of the dam, but a single berm at the base of a gorge will often be found useful. Generally, its height and top width should each not be less than one-quarter the height of the dam.

FIG. 3

SECTION

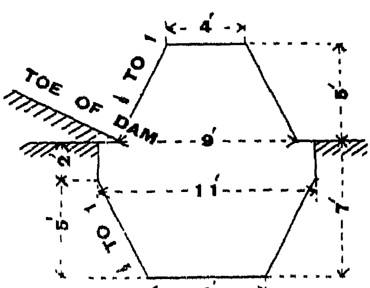
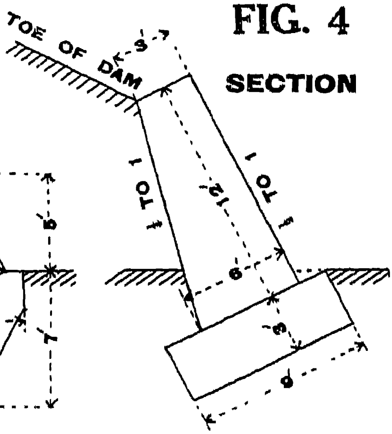


FIG. 4

SECTION



77. Toe Walls.—A toe wall is another device with the same object in view as a berm at the base to prevent motion at the toe. Fig. 3 shows a form which is meant to secure the foundation of the dam and not its superstructure. A trench is excavated through the unreliable foundation and is filled with drystone which is weighted by the superstructure of the wall.

[1] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii., pp. 219, 222, and 226.

Fig. 4 shows a form (really a retaining wall) which is meant to buttress the toe of the dam in the same way as does a berm. In all cases where earthwork has thus to be supported, it is best to make the section of the wall batter on both sides to the embankment, as shown, for it then offers an active resistance to the motion of the earthwork, and not merely the passive one of its stable weight. Upstream of the wall a drystone casing should be laid to collect the internal drainage of the dam, and this should be passed out of the wall by one or more slab drains, or large weep holes. The drystone toe described in paragraph 128, p. 178, may be considered to be the ultimate development of a toe wall.

78. The Relative Cost of the Dam and the Waste-Weir.—The height of the dam depends upon the high-flood level of the reservoir, and this again upon the discharging capacity of the waste-weir and its crest level. The longer the waste-weir is made, the less will be the depth of the high-flood over it and the lower need be the dam. Comparative estimates of the dam of different heights and the weir of different lengths should be made in order to see which is the cheapest combination, remembering (para. 166, p. 219) that the longer the weir, other things being equal, the safer the work.

Again, the level of the saddle, or ground in which the weir is to be formed, may be higher, or lower, than what the full supply-storage capacity requires. It will generally be advisable to have the weir crest at the level which will most fully utilise the total yield from the catchment. When, however, the question arises of cutting down the ground extensively at the weir site, comparative trial estimates of the weir and

the dam together should be made so as to see at what full-supply level the cheapest rate of storage can be obtained. The heightening of the dam does not appreciably affect the cost of the outlet, nor add to that of the buildings and the general preliminary charges, while the enlargement of its section will generally be less in proportion than the increase in storage contents due to raising the full-supply level. It is usually better to have a storage capacity slightly too large than one which is too small for the catchment.

IV. Materials for Dam Construction.

79. Selection of Material necessary.—The material of which a dam has to be formed requires careful selection: on the one hand, it has to be water-tight; on the other hand, while possessing adhesion, it should offer resistance to slipping. Powdery, dry material which will not bind, such as some kinds of marl (limy soil), light loose material such as peat, and soils which are impregnated with salt or turn into slush by the action of water, or which when dry break into fragments with sharp angles and smooth, shining surfaces, should be rejected. Pure sand has, however, been used for some dams, as it has the property of settling into a compact mass when wetted, and the angular form of the particles gives it a considerable amount of resistance to slipping. Sand in combination with clay is, however, not good, as it admits water into the latter material but does not allow that to drain out, while its particles, being so fine, do not add much frictional stability to the clay in which they are embedded. Pure black "cotton-soil" and rich clayey earths are dangerous, as, when wetted, they become

greasy and treacherous, and are thus particularly liable to slips; they are, moreover, very retentive of water. The best material is one containing enough clayey matter to enable it to bind, and thus become water-tight, and enough shaly matter to give it frictional resistance to slipping and the property of self-drainage, so that the whole mass never becomes sodden. The best natural soils are those which when dry break into tough, not brittle, fragments and have a dull and irregular fracture. When any doubt arises as to the suitability of the soil proposed to be used, small trial tanks should be made with it and should be filled with water; the behaviour of their banks should be noted when their earthwork is saturated and again after it has been allowed to dry. If the surface of the banks cracks during desiccation, it is evidence that more shaly matter should be mixed with the soil.

80. The Disposition of Material in the Section.—When the earliest dams were constructed in Bombay by British engineers there was a fear that the introduction of any material not itself water-tight would lead to infiltration of water, and thus be harmful to the stability of the work. Pure black "cotton-soil" was therefore used throughout the section (Fig. 5 on page 117), and it was specified that all stony particles of any size were to be removed.

The next step taken was to use this soil in the centre of the dam to form an impervious hearting; to weight it, and to prevent it from slipping, it was confined in place by side sections of "selected material" of heavy, shaly soil. On the upstream side this material was of an impervious nature, so as to resist the infiltration of water, and, on the downstream side, of a more shaly character, so as to resist any tendency to slipping.

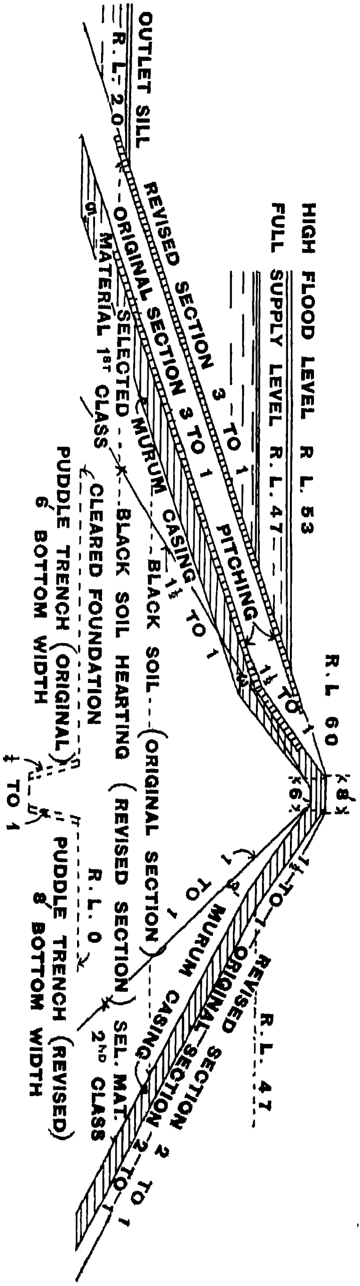


FIG. 5

ORDINARY TYPE SECTIONS OF DAM 60-FT HIGH

This section is also sketched in Fig. 5. It will be seen that it is virtually of the English type, the puddle wall of which is replaced by a much thicker clayey hearting of a less retentive nature than the puddle clay that is procurable in England, but which can seldom be had in India.

The final step taken, which is the one strongly recommended for adoption, was to use a soil of the same character uniformly throughout the dam, but, instead of depending solely upon its water-tightness, to select one that was both impervious and stable under the action of water: such a soil is described in the middle of paragraph 79, p. 116. There are natural soils which answer to that description, but, where they do not exist within an economical distance from the site of a work—say half a mile—an artificial mixture should be made instead. The proportions recommended are 1 of pure black "cotton-soil," or other clayey soil, to 1 of pure muram, or shale. Where these materials are not found in a pure state, the existing soils should be mixed in the proportions which will result in the production of a similar mixture.

It must be remembered that, although the upstream slope is saturated by the water of the tank, it is adjusted to meet that saturation, and is under fairly settled conditions, for the water in the reservoir will not, as a rule, rise very rapidly during the monsoon, and it will fall very gradually owing to the draw-off during the fair weather. Moreover, as far as it is submerged, that slope is supported by the water in the reservoir. The downstream slope is much steeper, but is under less settled conditions, as it will become considerably

and suddenly saturated during the monsoon downpours and less damp during the intervals between them, while in the fair weather it will be much dried. To meet this difference of conditions it is not unreasonable to form the downstream slope of material equally good and as carefully made as that of the upstream slope.

The great advantage of having the section of the dam homogeneous is that it will act uniformly as one mass during the process of settlement and self-consolidation, (which will last some years); thus will be avoided the formation of internal stresses, owing to the different rates of settlement of its constituent parts which would occur if the section were made of different materials. A difference of settlement might cause the formation of a slip, and will, anyhow, affect the disposition of the layers in which the dam was originally constructed. There is also a practical consideration to be taken into account in regard to this method of construction. It is easier to have a uniform material throughout the section than one varying in different parts.

81. The Casings of the Dam.—The casings of the dam may be considered to be apart from the main dam. Owing to their narrow width they will not affect it as a whole, and their object is a special one, namely, to protect the interior from external influences. On the upstream side it is necessary to form a firm and insoluble foundation for the pitching, and, on the downstream side, a covering which will not crack when dried by the sun nor gutter when subjected to rainfall. As the pressure of the reservoir water on the upstream side and the scouring action of rainfall

on the downstream side increase directly as the height of the dam, the casing should be wider at its base than at its top. The widths, measured normally to the slopes, recommended are :—

At the top of the dam . . .	2 feet.
From the top of the dam to high-flood level on both sides . . .	2 feet.
Below high-flood level on both sides :— increasing uniformly at the rate of 1 foot in 10 feet vertical.	

The material of the casings should consist of a mixture equivalent to 1 of pure argillaceous soil to 2 of pure shale. The casings should be constructed uniformly with the hearting so as to be thoroughly bonded with and united to it, and should not be patched on subsequently.

82. The Utilisation of Spoil.—An economical advantage of mixing gritty material with the ordinary argillaceous earth is that all sound spoil from the excavations can thus be utilised in the formation of the dam. Where this grit exists in large quantity beyond what is required to form the prescribed mixture, the excess can be incorporated with the material forming the downstream part of the dam, and the more stony portions can be reserved for the lower part of the downstream slope. The only spoil which should be rejected for incorporation in the dam is powdery, peaty, sandy, salty, or slushy material (para. 79, p. 115).

83. American Practice.—In American practice a still greater use of gritty material obtains. Mr. Fanning[1] recommends the following proportions for dams :—

[1] "Hydraulic and Water Supply Engineering," 8th edn., p. 340.

	By volume		Per cent.
Coarse gravel . . .	1·00	cubic yard	59
Fine gravel . . .	0·35	,,	20
Sand	0·15	,,	9
Clay	0·20	,,	12
Total when loose . .	1·70	,,	100
Total when consolidated.	1·25	,,	

This mixture, when properly consolidated, would be free from voids, the proportions being adjusted for this purpose, but the small amount of clay used would apparently not make the mass sufficiently impermeable, and would prevent it from possessing much cohesive stability. Mr. Clemens Herschel[1] distrusts the use of clayey material, and would not bring it on to the site of the works: he prefers a gravel that would puddle, or "binding gravel." To test the suitability of such a gravel for the construction of a dam, he would mix it with water in a pail to the consistency of moist earth as generally used in a dam. If on turning the pail upside down the gravel remained in the pail, it would be of the right character for use; but if it dropped out, it would be too gritty for employment, and should be rejected.

V. The Puddle Trench.

84. The Object of the Puddle Trench.—Practically no subsoils are watertight, especially under the great pressure of water due to its storage in a reservoir. Ordinary earths may be of naturally porous material, or if of comparatively water-tight material, may exist in layers between which water will find its way. Even

[1] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii, p. 253.

when compact and clayey, the upstream particles, under the great pressure of the reservoir, will become charged with water, and will thus wet the ones downstream of them, so that gradually there will be a more or less slow passage of water through them. Rocky soils are very pervious if their particles are not cemented together to form a water-tight mass. Many rocks are stratified, and the spaces between the layers are either open or are filled by dry, porous material: in fact, the only water-tight formation is one of dense, unfissured rock. The object of a puddle trench is to interpose a water-tight *septum* which will tend to prevent the passage through it of the water coming to it, and thus will tend to keep all the material downstream of it as dry as possible.

85. The Position of the Puddle Trench.—Taking only its water-intercepting object into account, the puddle trench should be placed as much upstream as practicable, so that the dry area downstream of it may be as large as possible. The limiting position in this respect is one where a sensible amount of infiltration through the superstructure of the dam will not pass over the trench and saturate the subsoil downstream of it. Another matter has, however, to be considered. The puddle filling, being of a different character to the natural strata through which the trench passes, and, usually, being originally of a more compressible nature, there will at first be unequal and generally greater settlement over it than on each side of it. For this reason the position almost invariably selected for the puddle trench is on the centre line of the dam, so that the settlement of the superstructure may be uniform on each side of that line. The greater weight of the superstructure over this line, where the height of the

dam is at a maximum, also acts beneficially in compressing the material in the puddle trench to the greatest extent. When a puddle wall is to be constructed, the puddle trench should be aligned vertically below it.

86. The Depth of the Puddle Trench.—It is essential that the puddle trench should intercept such underground flow as would in the course of time carry away particles of the subsoil, thus enlarging the water passages and eventually causing the undermining and destruction of the dam. As it is not practicable after the completion of the embankment to make the puddle trench secure, it must be made quite safe at the time of its construction.

A less important consideration is that the puddle trench should prevent the leakage away of so much storage as would sensibly diminish the utility of the reservoir. With a properly constructed trench of ordinary dimensions, this amount of loss is, however, not likely to occur originally and will probably decrease as the reservoir bed gets silted. It can be compensated for usually by a slight increase of the full storage level, seeing that the storage contents at the top contour of a reservoir are greater than those at the one immediately below it, and much greater than those at the lowest contours, while the increase of pressure, due to the small increase of level required, is practically inappreciable. In many cases also—such as where the canal or a minor channel is led off below the dam from the stream which supplies the reservoir—the water lost by leakage will be picked up by the weir forming the headworks of the distribution system.

The depth to which the puddle trench should be

excavated depends principally upon the porosity of the soil through which it is carried, and should be made less for compact soils and greater for porous ones; it also depends greatly upon the head of water in the reservoir which it has to resist (para. 64, p. 92). Assuming that good, compact soils are met with, the depth of the puddle trench need not ordinarily exceed one-half the depth of the full-supply storage; and that the soils are fairly compact, not more than the depth of that storage. The high-flood depth need not be considered in this connection, as the duration of floods will usually be too short to increase the amount of subsoil flow to, or below, the puddle trench. The minimum depth of the trench at the flanks when sound rock is not met with should be 6 feet.

When the surface of sound, unfissured rock is near ground level, or not much below the depths above noted, the trench should be carried at least 1 foot into it, as there will nearly always be a considerable amount of leakage along the junction of the rock with the top-lying soils. If the rock is much fissured, the fissured parts should be cut out: it may not be necessary to remove these for the whole width of the trench as a small trench may be excavated through the fissured layers along the upstream side or centre of the bed of the main trench, and this should be filled with puddle, or concrete in bad cases (Figs. 7 and 8, p. 137). When, however, the rock lies at a much greater depth, the trench may be founded at the depths proposed above, provided it be carried at least 2 feet into good, compact and water-tight clayey soil, extending for some feet below its bed. If such a foundation is not met with, the trench must be

excavated deeper. It must be made to pass through all sandy and highly pervious layers, and, if these exist to a great depth, the idea of making a reservoir at the site may have to be abandoned.

Where the dam is situated on a narrow ridge of porous soil, or fissured rock, both the width and the depth of the trench should be increased to cut off the greater subsoil flow which may be expected there.

The nature of the subsoil is determined by sinking trial pits through it on the centre line of the dam. If these disclose a fairly regular disposition of the subsoil, they may be spaced up to 500 feet apart. If, however, that disposition is irregular, the trial pits should be nearer together, so that the levels of the strata may be correctly determined and plotted on the longitudinal section of the dam. The trial pits should invariably be carried down to sound-rock level, so that its position and the nature of all the soils above it may be ascertained with a view to the formation of a decision as to how deep the puddle trench should be carried from point to point, and whether the foundation is good enough for carrying the dam.

Trial pits should always be excavated as near as possible to where the main and minor drainages cross the dam line, as there the foundation may be less reliable, the strata may vary more rapidly, and the puddle trench may have to be carried lower than is necessary at adjacent places. As bore holes, owing to their small diameter, give less reliable information than do trial pits, they should not be adopted in preference to the latter for investigating the nature of the subsoils.

87. The Bottom-width of the Puddle Trench.—The bottom-width of the puddle trench depends upon :—

(*a*) The nature of the strata passed through, and especially of those near its foundation level;

(*b*) the nature of the filling and the amount of consolidation to be given to it at the base of the puddle trench;

(*c*) the depth of the full-supply storage at the point and the importance and size of the reservoir.

Where the bottom strata are porous, the width must be made greater than would suffice were they compact. Where the total depth of the trench is small, the bottom-width must be increased beyond what is sufficient for a deep trench, so that at ground level the top-width may in both cases be equally sufficient to resist the greater percolation probable there.

For large works the puddle trench filling should be of a more retentive quality and should be more consolidated, than is necessary in the case of small works: for the former it is essential that the filling should be consolidated by rolling; for the latter, ramming will be sufficient.

Where the depth of the full-supply storage at the point is great, the amount of storage is considerable, and the reservoir is important, the width must be made greater than it need be where the conditions require less care being taken to prevent leakage.

As a general guide it may be said that the bottom-width of an important puddle trench should not be less than 10 feet, so as to give space for a roller to work, while for a small work it may be reduced to 6 feet. A smaller width than 6 feet is of little practical use in cutting off leakage. Subject to these restrictions, the base-width may be made equal to one-quarter of the full-supply depth of the reservoir at the point considered; another rule for it is that it

should not be less than one-eighth of the full-supply depth *plus* 3 feet.

88. The Side-slopes of the Puddle Trench.—The side-slopes of the puddle trench have to be determined by two considerations :—

(*a*) They should be flat enough to stand during excavation and until the filling of the trench has been completed ;

(*b*) They should be flat enough to give the puddle filling a width increasing sufficiently as it rises towards ground level to enable it to resist the greater infiltration it will have to withstand at the higher parts of its section.

In respect to (*a*) it may be noted that, as the time the trench will be open will generally be short, the slopes may be excavated steeper than would be necessary were the excavation to remain permanently open. They should, however, not be made steeper than $\frac{1}{4}$ to 1 in soil, nor than $\frac{1}{8}$ to 1 in rock ; at the former inclination the top-width of a trench 30 feet deep and with a base-width of 10 feet would be 25 feet.

There are certain formations where sandy pockets, or layers, occur irregularly. Where these are numerous, it is advisable to widen the trench on the upstream side, 5 feet or more, and, as the filling rises, to pick out the sand from the pockets and layers, as far as they can be undermined with safety, and at once to fill them with puddle material.

89. The Length of the Puddle Trench.—At the full-supply margin of the reservoir the water pressure at the surface is practically *nil*, and a considerable depth of trench there is generally unnecessary. Nor need the trench be extended longitudinally for any great distance beyond full-supply level, as floods will

usually be short-lived, and therefore will not have time enough to develop much additional leakage. As the slope of the country will generally be flat at this level, it will, ordinarily, suffice not to continue the trench beyond the high-flood contour. In England, in certain cases, the trench is made much longer, but that must be because the subsoil is more stratified there than it usually is in India. If a porous layer extends beyond the high-flood contour at a level below full-supply, it would lead to the out-flanking of the dam if not cut off by an extension of the puddle trench; in such a case, the trench should be continued to prevent this.

90. The Height of the Puddle Filling.—The puddle material is usually carried up—say, 2 feet—above the natural surface of the ground so as to prevent the formation at this level of a defined line of flow across the base of the dam. This extra height of filling should be constructed at the same time as the embankment on each side of it, and should be rolled and consolidated with the latter, the only difference between the two parts being that of the materials of which they consist. If the completion of the puddle trench has to wait for the construction of the dam, the upper surface of the puddle should be kept a few inches below ground level for convenience of work until the embankment can be taken in hand. When the filling can be continued and completed, this temporary surface should be removed, until all cracked or loose material has been taken out, and then the remainder of the puddle work should be finished and at once covered over with embankment for its protection.

91. The Continuity of the Puddle Trench.—If the

puddle filling is disturbed so that a leak is formed through it, its efficiency will be lessened, and, as its material is of a naturally soluble character, the leak may possibly increase so as to become dangerous. The puddle being of a compressible nature, any sudden change in its section may cause unequal settlement and hence a leak. It is therefore essential that such changes should be avoided, for which reason the bed should never be stepped up abruptly, but all changes of its level should be effected by gentle slopes. Similarly, there should not be any projecting shoulders left in the side-slopes, as these may prevent the filling, during settlement, from occupying all the section below them; if this happens, spaces may be left in which the infiltering water may attain hydrostatic pressure sufficient to allow it to force itself as a defined leak through the puddle.

In short, the trench, both in longitudinal and in cross-section, should be bounded by slopes, and thus be wedge-shaped, so that the only result of the settlement of the puddle will be to make it more compact and fill the excavation more completely.

92. Filling the Puddle Trench.—The bed of the trench should first be roughened so that the filling may be thoroughly bonded with it and a defined line of flow between the two may thus be prevented; for this purpose the upstream half of the trench may be excavated as a wedge, say a foot deep below the downstream half (Fig. 7, p. 137). The surface of the bed should be cleared of all dry material, and, when clean, should be wetted to receive the filling. If the trench is carried into rock, all fissures should be cleared out, or grouted, and the surface of the bed should be washed. The bottom layer of the filling, about 1 foot

thick, should be formed of carefully kneaded balls of clay thrown forcibly on to the bed and then thoroughly trodden so as perfectly to unite with it.

The subsequent filling should be carried out in layers, which, when completed, should not exceed 3 inches in thickness. These should be so formed that the whole mass will be quite free from stratification and leakage planes; this can be avoided by wetting the surface of the completed layer just before the new one is laid on it, so that the material of the latter may be forced for a small depth into that of the former and may thus be perfectly united with it. As an additional precaution, at vertical and horizontal intervals, and breaking joint with each other, small wedge-shaped trenches, say 3 feet wide and 1 foot deep, should be excavated through the completed work, and should be refilled and consolidated just before the fresh layer is added. Further, to offer as much resistance as possible to the passage of water through the material, the layers should be tilted as steeply as practicable, say 1 in 6, rising from the upstream side of the trench.

The material of the puddle trench filling should be the most retentive clay procurable within half a mile of the site. A mixture with it of gritty stuff would make it more permeable, for, even if that material is itself water-tight, it and the clay will not be in perfect union, and water will tend to find its way between the two and will have a shorter course from one stony particle to the next than it would have through a homogeneous mass of solid clay. In the superstructure of the dam a mass of pure clay is objectionable, as it may slip, but this it cannot do in the trench, for there it will be supported by the sides of the excavation; nor can it be forced out, as it will be kept down by

the weight of the embankment. In the dam itself a mixture of gritty material is useful in preventing shrinkage and the formation of cracks, in bonding the layers together, and in permitting the self-drainage of the earthwork so as to let it acquire greater stability. In the puddle trench such a mixture is not required for these purposes, as there the filling is under settled conditions and should therefore be made as impermeable as possible. The filling should be constructed so quickly that no drying and cracking of it can take place, and the layers should be united together as above explained.

The filling material should be deposited as dry as is consistent with its being thoroughly consolidated. As explained in paragraph 63, p. 91, the wetting of clay causes it to expand, and any original moistening of it, beyond what is necessary to cause it to bind, will make it less dense and more permeable, and also will diminish its power to support the dam (para. 85, p. 122). The wetting that puddle (which was originally made with just a sufficiency of water) receives by percolation from the reservoir, subsequently to the completion of the dam, will cause its particles to swell and thus render the whole mass more compact and less permeable.

The filling of important trenches should be consolidated as much as possible by rollers. Further consolidation will result from the superincumbent weight of the dam, and this should therefore be allowed to act for as long as practicable before the reservoir begins to fill, so as to prevent filtration through green material.

Before the filling of the trench is commenced, all springs in it should be carefully led away in pipes, or

otherwise, and, when the filling has reached a sufficient height, the pipes should be securely plugged.

In Appendix No. 19, p. 438, are given tables for the calculation of the excavation and filling of puddle trenches.

93. Concrete Trenches.—The above paragraphs have dealt with earthen puddle trenches; but there is another class of an impervious *septum*—one made of concrete—which is often adopted in England, although it is seldom, if ever, constructed in India on account of its expense. For concrete trenches the excavation is taken out in narrow timbered trenches, which in some English examples have been sunk as deep as 212 feet[1] below ground level. Such an extreme depth is justified only when the value of the impounded water is very great, *e.g.*, as in the case of a water-supply scheme, or where the lower strata are very porous and would otherwise constitute a danger to the dam. It is doubtful if even such a great depth would entirely prevent the passage of all water, although it would be useful in affording, at a low level on the downstream side, a large extent of natural drainage area for passing off harmlessly the lessened amount of percolation water which had got through the filling of the trench.

The thickness of a concrete trench depends partly upon the width necessary for its timbering, and partly upon the porosity of the soil passed through. The objection to a concrete trench is that, as it is formed in layers, it is apt to be stratified. To guard against this the surface of an old layer should be roughened and should receive a thin coating of mortar before a new layer is formed on it. The concrete should not be

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[1] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii., p. 207.

thrown down from a height, as then the aggregate will separate and fall to the bottom of the layer and make it porous, but it should be carefully lowered and spread on the completed layer and should then be well rammed so as to fill the trench completely. The concrete should, moreover, be rich in mortar, the aggregate should be fine, and the layers should be laid in thin courses—say 4 inches thick—in order that they may be consolidated uniformly throughout, and one course should be constructed at once on top of the other, so as to become perfectly united with it. As many courses as practicable should be made at the same time to form a single layer and thus to reduce the number of horizontal leakage planes.

In India concrete would cost nearly eight times as much as puddle, and the excavation of a timbered trench, about as many times that of an open one. Although the clayey soil usually obtainable in India is not so good as the puddle clay procurable in England, still the much greater thickness economically permissible with it renders it generally as safe to use as concrete.

94. Concrete Key Trenches.—While a concrete trench is seldom constructed in India instead of a puddle trench, one may, with advantage, occasionally be made at the base of the main trench to supplement it. If a hard pervious soil exists there, a narrow trench may be excavated in it and filled with fine concrete; the concrete should be carried up, say, 2 feet, above the bed of the main trench, so as to key into its puddle filling, care, of course, being taken properly to consolidate the projecting key (Fig. 8, p. 137). If there is a layer of fissured rock at the bed of the puddle trench, it should be removed and may be replaced

by a layer of concrete across the whole bed, with a concrete key constructed on top of it (see also para. 86, p. 124).

95. The River Crossing of the Puddle Trench.—Not only is the river crossing the deepest part of the dam foundation, but also it will probably have the most fissured subsoil; extra precautions to cut off leakage should therefore be taken at it. For a high dam these had best assume the form of a thick wall, with masonry facings and a hearting of fine concrete, carried up for some height into the body of the dam and continued laterally for some distance as sloping ramps into the flanks. There the wall should be keyed into the puddle trench, which should be widened out to overlap it on both the upstream and downstream sides. The foundation of the wall should be taken down into a perfectly sound, hard, and water-tight stratum, and preferably into rock (Plate 5, Figs. 2, 5, 6, and 7).

In such cases where it is necessary to make a long water-tight junction between masonry and earthwork, the face of the former should be free from projections, and may, indeed, be plastered over with mortar or luted with clay. Water is said to have an abhorrence to a right angle, for which reason small projecting staunch pilasters should be built at intervals along the upstream face of the wall, and intermediately should be small key recesses, so that the masonry and the earthwork may be joggled and united together.

On the downstream side of this concrete wall should be a continuous drain formed in the usual way and leading to the main rear drain (para. 108, p. 152).

Where a minor drainage is crossed, the puddle trench should be widened and deepened, and, if

necessary, a concrete key trench should be constructed below it.

96. The Drainage of the Puddle Trench.—Notwithstanding all precautions, some water is likely to pass through the puddle trench. Certain English engineers dispute this, and state that the puddle trenches of their works are absolutely staunch. Probably leakage does occur through them, but is not directly apparent, as it may be carried away unperceived through porous lower strata, and may not appear on the surface for some distance from the sites of the reservoirs. Hence, when the subsoil is pervious, the drainage of the puddle trench is not so necessary as when that is fairly impervious. In Indian reservoirs there is no doubt that underground percolation does occur; the level of the water in wells downstream of them is raised, and clear springs issue at rocky outcrops below the works. Compared to the great length of the dams and the great pressure of the storages, the amount of loss from this source is very small, but still it takes place. In India the objection to leakage is not to its amount, which can be compensated for by a small increase to the contents of the reservoir (para. 86, p. 123), but to its effect in soddening the area below the dam. In a well-constructed dam, owing to its great thickness, there should not be any percolation through it, and the actual junction of the base with the natural ground can also be made staunch owing to the considerable length through which the leakage has to find its way.

The case is different with the puddle trench. Compared with a dam 60 feet high, having a base-width of, say, 310 feet, a puddle trench 30 feet deep may have a top-width of only 25 feet, or only one-twelfth the

width of the base of the embankment, and it is enclosed on both sides by material of a more or less porous nature. The filling of the trench is certainly not twelve times as staunch as the material of the superstructure of the dam, and it would therefore seem reasonable to attribute the leakage which occurs below a reservoir to percolation through the puddle trench.

The questions to be decided are :—

(1) Should this leakage be allowed to sodden the area downstream of the dam ? or, should an attempt be made to lead it away harmlessly ? and

(2) Can it be drained away without inducing a still greater amount of percolation, which may endanger the stability of the work ?

It is believed that the answers to these questions are that :—

(1) The soddening of the area below the dam is at least undesirable, even if it is not distinctly dangerous, and endeavours should be made to prevent it ; and

(2) The puddle trench may be drained with safety and without inducing a greater flow through it.

97. The Construction of the Puddle Trench Drain.—Figs. 6, 7, and 8 show three forms of puddle trench with drains adapted to meet the conditions described therein. In each case a drystone drain is shown with a vent of 6 inches height and 4 inches width, which are the minimum dimensions at the head of the drain : in a long puddle trench the vent might be increased gradually to 9 inches square at the outfall of the drain. The minimum thickness of the sides of the drain should be 9 inches, and that of the slab covering, one varying from 6 inches for a 4-inch vent to 9 inches for a 9-inch vent. Round the drain should be a ring about 18 inches thick, of sound, clean, and sharp

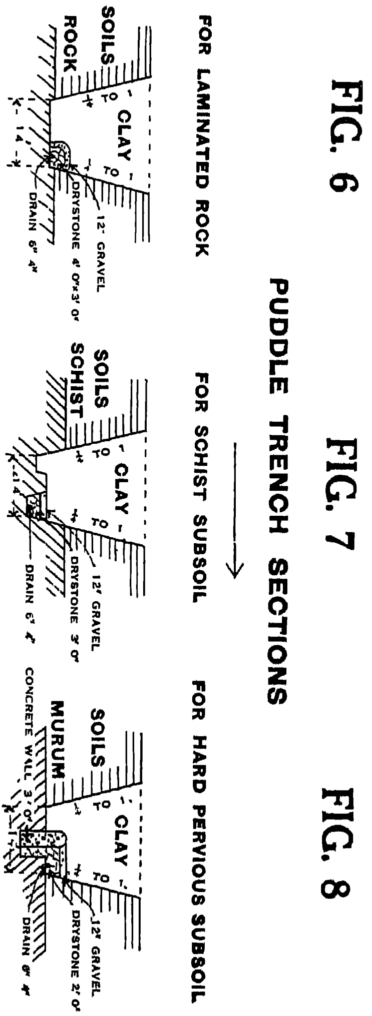
FIG. 6
FIG. 7
FIG. 8
PUDDLE TRENCH SECTIONS
FOR LAMINATED ROCK
FOR SCHIST SUBSOIL
FOR HARD PERVIOUS SUBSOIL
SOILS
ROCK
SCHIST
MURUM
CLAY
12" GRAVEL
DRYSTONE 4' 0"×3' 0"
DRYSTONE 3' 0"
DRYSTONE 2' 0"
DRAIN 6" 4"
CONCRETE WALL 3' 0"

small rubble; this should be surrounded by a ring about 9 inches thick of clean gravel, small quarry spauls, etc., and that by a ring about 6 inches thick of clean, coarse sand. The object of these rings is to enclose the drain by filtering material (through which soil cannot be carried), so that the vent may always be clear.

The bed of the drain should be solidly founded to prevent any settlement, and, if not of rock, should be formed of through slabs laid in mortar: the sides should be made of long stones laid in mortar with narrow dry joints on the upstream side at intervals, of say 5 feet, to act as weep holes; the top should be of through slabs laid in mortar, with narrow open joints at similar intervals intermediate with those of the sides. The drain should slope continuously from its head to its tail, and no part should be lower than any portion of the length downstream of it. Where the puddle trench has a dip, the bed of the drain should be carried across it at its own regular inclination and should there be supported by a masonry or concrete foundation. The low part of the bed of the trench itself, at such a dip, can be drained upwardly by means of an iron pipe, having its outlet end carried up so as to discharge into the drain at its roof, and built in in masonry to secure permanency to the vent (Elkington's system, para. 108, p. 153). Or, better still, the whole of the low part of the whole bed of the puddle trench should be excavated to a narrow section and filled to the level of the bed of the drain with concrete, (so as to raise the subsoil flow to it), and a small concrete key should be formed on top of the drain.

To prevent, during construction, the clogging by

earth of the rings of dry material surrounding the drain, the clay filling of the base of the puddle trench should first be constructed for a height of about 1 foot above the top of the future sand cover of the drain, and this will enable that filling to be consolidated thoroughly. The space to be occupied by the drain and its filtering cover should then be neatly excavated, the drain built and surrounded by the filtering material, and covered by at least 1 foot of clay carefully rammed; thereafter, the filling of the whole trench should be resumed for another 2 feet, giving, temporarily, a total earth cover of 3 feet above the filtering materials.

Before the trench filling is raised any higher, it is desirable to complete the whole length of the drain and to test its freedom from obstruction by passing water down it. Similarly, it may be advisable to finish off the upstream end of the drain by building it up along the upstream end slope of the puddle trench there, and then to carry the drain with a gentle upward slope out of the dam to a closed cistern, so that, if necessary, it may be tested and flushed with water at any time after the completion of the embankment.

98. Infiltration not induced by the Puddle Trench Drain.—The construction of the drain necessitates the widening of the base of the puddle trench to a minimum width of, say, 14 feet, which, of course, entails so much extra expenditure, but is otherwise an advantage, as the wider trench will be all the more water-tight. As the drain will be formed at the downstream edge of the puddle trench, there will be a considerable thickness of puddle between it and the upstream edge; and there is not any reason why it should induce an excess amount of percolation. On the contrary, it

should keep the puddle filling above it well-drained and thus render it more compact and impervious. In a well-constructed puddle trench the subsoil water will tend to descend along the upstream face of the puddle to the base. If it does not there meet with a means of escape, it will endeavour to force itself up, thus rendering the filling less compact and more pervious. The drain, by leading away this relatively small amount of percolation, provides the required means of escape, and thus tends to prevent the puddle filling from becoming deteriorated and the base of the dam from getting sodden. In England, when springs are met with, they are generally led out of the trench by means of pipes, which is an arrangement for securing drainage at definite points. The above-described drain is designed to intercept percolation water throughout the whole length of the puddle trench and to pass it safely out of the dam.

The drain should not be continued as a drystone casing all the way up the downstream face of the puddle trench, for such a long, thin layer might suffer distortion or disturbance from the lateral pressure of the puddle or of the earthen sides of the trench. Thus it might become discontinuous and therefore harmful by collecting water, which might subsequently force its way out and form a leak through the puddle. In the case of a concrete trench in hard soil, this dislocation would not occur, and the concrete with advantage might be backed by a rubble lining to act as a drain (Fig. 8, p. 137).

99. Puddle Trenches not suitable for Famine Work.—The excavation and filling of even ordinary puddle trenches have to be executed with comparative slowness, and to be carefully supervised; more especially

will this be the case if base drains have to be formed. Such work is, therefore, not suitable for the employment of large bodies of famine relief labourers (end of para. 113, p. 162). Where the construction of storage works has been set aside for relief purposes, it may therefore be advisable to complete the puddle trenches in advance. The filling should in such cases be carried up, say 2 feet, above ground level to protect it from being dried, and this 2 feet should be constructed of a fairly porous mixture to permit of the infiltration of rain water, which will keep the filling moist. When the dam has to be commenced, this 2 feet and any underlying part which may be dried and cracked should be removed and replaced carefully by puddle material.

100. Nulla Puddle Trenches.—If the dam is on a sharply-defined ridge line, there may be at the base of the ridge a nulla, parallel to the dam and close to it, which may disclose porous strata below the level of the bed of the main puddle trench, and these would, if not dealt with, allow leakage water to pass below the trench. In such a case it is advisable to form a subsidiary puddle trench along the downstream edge, or side next the dam, of the nulla (Plate 4, Figs. 1 and 2, and Plate 5, Fig. 3).

When the reservoir fills, its water will pond up along this nulla bed and retard the flow of the nulla water, so that fine silt will be rapidly deposited in the deeper part of the bed, *i.e.*, that nearest the main stream, and will puddle it with fairly water-tight material.

101. Proposed Concrete Trench for the Bohio Dam, Panama Canal.—Mr. J. T. Ford,[1] M.Inst.C.E., has

[1] *Engineering News*, Vol. xlviii., pp. 377–9, November 6th, 1902.

PROPOSED BOHIO DAM — PANAMA CANAL

FIG. 9 LONGITUDINAL SECTION

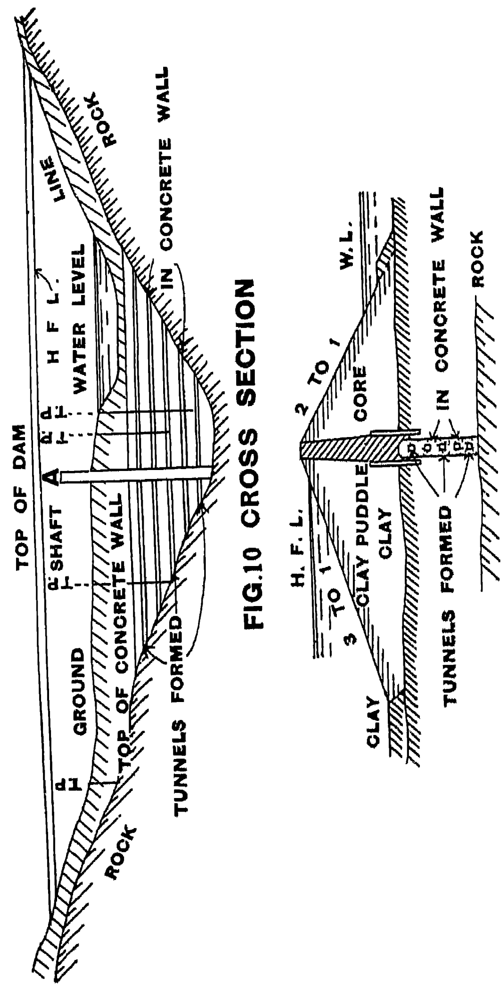

FIG. 10 CROSS SECTION

described a novel and ingenious arrangement for the construction of a concrete trench for an earthen dam across the large Chagres River, which is liable to bring down enormous floods. Such a trench may be 128 feet deep below the low-water surface of this perennially flowing river. Stated shortly, he proposes (Figs. 9 and 10) to sink a shaft A on one bank beyond the reach of ordinary floods; to carry it down to bed rock; to build it above the reach of extraordinary floods; and to follow the bed rock on both sides from it by tunnelling (freezing the water-bearing soil, if necessary), and by adopting the shield form of excavation, or, if the conditions permit, by means of ordinary trench timbering. The surface of the sound rock having been cleansed, the tunnel is to be filled with concrete. After the tunnel at the bottom has thus been completed, another one is to be excavated above it, and is to be filled similarly in perfectly water-tight connection with it. The trench is thus to be gradually completed by a series of similarly constructed layers in tunnels. The top-section, replacing the bed of the river, might have to be carried out in open excavation by means of ordinary caisson and pneumatic work, sheet piling, cofferdams and dredging. When filling the successive tunnels, open culverts are to be left in them for the purpose of inspection at any time after the completion of the work. He also proposes to make an ordinary puddle wall on top of the trench, but a masonry core wall would seem to be the better form of superstructure for such a foundation.

The expense of such a construction would be so great as to prohibit its use in India, except, perhaps, in works of equal magnitude presenting like difficulties. It is described as an ingenious proposal for meeting

the conditions which exist at the site of the contemplated work.

102. Excavation in the Reservoir Bed.—In order to prevent percolation under the dam as much as possible, excavation should not be allowed within the areas immediately next its toes. The minimum width of these areas, on the upstream and downstream sides respectively, might be four times and two times the height of the dam at the point considered. Further, the entire stripping off of the water-tight cover of pervious strata on the upstream side should be prohibited within a minimum width of ten times the height of the dam, measured from its upstream toe. Although such excavations will soon silt up, they will not become water-proof for many years, and, until they thus again become staunch, they may cause excess subsoil filtration, which may act prejudicially on the newly-formed dam.

VI. The Puddle Wall.

103. Central Puddle Walls.—In England it is the general practice to make a puddle wall along the centre line of the dam and vertically over the puddle trench, so as to form with it a water-tight *septum* (extending throughout the dam from below bed-rock level to above high-flood level), in order to intercept any infiltration that may have penetrated so far. The following are the sections[1] which have been adopted for the puddle wall for certain works :—

(*a*) Dale Dyke dam—top-width, 4 feet; batters, 1 in 16, giving a base width of 16 feet where the dam is 96 feet high; this section is very light.

[1] *The Engineer*, Vol. lxiii., p. 189.

(*b*) Vehar dam, Bombay waterworks (in this particular not a typical Indian dam)—top-width, 10 feet; batters 1 in 8.

(*c*) The Bann and Harelaw dams—top-width, 8 feet. Rankine[1] states that the thickness of the base of a puddle wall should be about one-third of its height, and that the thickness of the top should be two-thirds, or one-half, of that of the base.

The objects of placing the puddle wall in the centre of the dam are:—

(1) To make it in vertical continuation of the puddle trench;

(2) To diminish its quantity to the minimum;

(3) To protect it from the action of the reservoir and the weather, and from damage by vermin.

The disadvantages of this position are:—

(*a*) It may be distorted or broken by unequal settlement of the embankment on each side of it, and during its own settlement, which will be considerable in amount;

(*b*) It is buried out of sight, and cannot be repaired for any considerable depth;

(*c*) The embankment upstream of it may be saturated to an undesirable extent.

The Board of Engineers[2] who examined the design for an earthen embankment for the Croton Dam, considered that a puddle wall would effect a drop in the hydraulic gradient of the line of saturation of 17 per cent. of the pressure head in the reservoir. They found the dams observed by them to be saturated as far as the puddle wall (*i.e.*, the hydraulic gradient

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[1] Rankine's "Civil Engineering," 11th edn., p. 704.
[2] *Engineering News*, November 28th, 1901, pp. 410–13.

was level up to this point), which would seem to indicate that this upstream portion had not been constructed sufficiently water-tight, and that the puddle wall tended to raise the line of saturation upstream of it (Fig. 1A, p. 98).

104. Slope Puddle Walls.—Another position in which the puddle wall has been placed is on the upstream slope of the dam so as to form a water-proof covering to it. The advantages claimed for the puddle wall in this position are that:—

(1) It will settle regularly with the upstream slope;

(2) Being on the surface, it can be repaired at any time when necessary;

(3) It renders the whole mass of the dam as dry, and thus as stable, as possible.

The objections raised to it are that:—

(*a*) It has no direct connection with the puddle trench, and thus subsoil water is not intercepted at it and may enter and soak the dam;

(*b*) It involves the construction of a greatly increased amount of puddle work compared with that in a central wall;

(*c*) It is liable to be washed by waves in the reservoir, to become cracked when exposed to the sun, and to be penetrated by vermin;

(*d*) It may not be able to stand at the ordinary slope of the dam.

In some works the puddle wall has been placed between the upstream slope and the centre line of the dam. In such a position its advantages and disadvantages are between those of the ones described above. The general opinion of English engineers is much in favour of the central position for the

puddle wall, but Indian and American engineers,[1] when they adopt a puddle wall, seem to prefer placing it on the external slope, in order to secure greater water-tightness of the dam as a whole.

105. Puddle Walls not now adopted in Indian Practice.—In modern Indian dams puddle walls have not usually been constructed on account of their disadvantages as described above. Indian engineers prefer to form the dam as one homogeneous whole and to make it solid and compact throughout, so that water cannot penetrate it to any great extent. The cheapness of labour in India permits of this being done there without great expense, but, as the compactness is obtained by rolling alone, thus thoroughly to consolidate the dam would not seem to be an extravagant precaution to take in any country; it will also secure the earthwork from the bad effects of an excessive amount of settlement. If, however, a water-tight *septum* is desired, it would appear better to adopt the masonry core wall (para. 58, p. 82) than the clay puddle wall, as it is less liable to failure, and because good puddling clay is not usually procurable in India.

The puddle wall is economical by limiting the amount of material to be made water-tight, but its use implies that the earthwork upstream of it is not reliable, and to have so large a mass saturated with water is most undesirable. The introduction of the puddle wall destroys the homogeneity of the dam, and this homogeneity is one of the principal objects which sound design and construction should secure.

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[1] Schuyler's "Reservoirs," p. 281.

VII. The Drainage of the Dam.

106. The Necessity for Drainage.—The proper drainage of the base of a dam is a matter of great importance, as, if arrangements for it are not made, it is possible that the embankment may subside, or slip, and instances of such failure have occurred. It is easy during the construction of the work to take such precautions as are necessary, but very difficult to effect perfectly reliable repairs after a dam has been completed. Drainage should be carried out before damage occurs rather than deferred until that has happened.

The maximum amount of percolation tends to take place along the junction of the dam with the ground, being derived, either from leakage from the reservoir finding its way along this course, or up through the subsoil, or from moisture descending from the heart of the dam itself until it is stopped by the foundation. The natural surface of the ground, being weathered and usually not having been subjected to pressure, is less compact than are the subsoils of similar constitution. The action of the sun in drying up the surface tends to produce cracks, which, in the case of "cotton-soil," are often visible for several feet in depth, and may extend still deeper. With other soils these surface cracks are not so deep, but they probably exist in all varieties to a more or less extent and facilitate percolation. The proper treatment of the actual surface is described in paragraph 113, p. 160.

107. The System of Drains Proposed.—The system of drains proposed is illustrated in Plate 4, Figs. 1 and 2, and Plate 5, Figs. 2, 4, 5, and 6, and is described below.

(*a*) *The "Surface Drain."*—There is no harm in dressing the surface of the ground at the rear of the downstream toe to a small depth provided this excavation is drained so as to prevent the formation of swamps. The general line of thrust due to the weight of the dam is inclined downwards, and the natural surface does not resist it as a buttress; it, of course, weights the subsoil and prevents it from rising, but the removal of a small amount of this weight is immaterial in this respect. It is far more important to prevent the whole of the subsoil at the rear of the embankment from getting sodden, for, if it becomes thus wet, it will tend to give under the weight of the dam. For this reason it will be best to slope off the ground just below the dam for a width of, say, 30 feet at an inclination of about 1 in 10 and with a longitudinal fall sufficient at once to carry away any rainfall running off the slope of the dam, even should vegetation grow on the dressed surface and retard flow.

This "surface drain" should be constructed in sections corresponding with the changes of the slope of the ground and not more than 300 feet long, each section being separated from the neighbouring one by an unexcavated strip, say 10 feet wide, which will prevent the formation of longitudinal scour channels.

The flow at the downstream ends of these sections should be diverted from the dam by small outfall gutters (which should be carried in water-tight channels over the "downstream drain"), and should be discharged some distance below the embankment.

(*b*) *The "Downstream Drain."*—Just downstream and clear of this "surface drain" should be a parallel "downstream drain", which may have a base-width of

5 feet, side-slopes as steep as practicable up to $\frac{1}{4}$ to 1, and a depth of from 10 feet to 15 feet; it should preferably be carried down to 1 foot below the surface of unfissured, sound rock, if this exists within a moderate depth. It should have a small drystone base drain with a vent, say 4 inches wide and 6 inches deep at the head, and, if the ground levels permit, should be divided into sections, separated from each other by unexcavated soil 10 feet wide, each leading out to the natural surface by outfall drains similarly constructed. If the ground levels do not allow of this being done, the drain should be continued until it can discharge into the river bed, the size of the vent being gradually enlarged to pass off the drainage as it increases in amount.

The trench excavated for the drain (which should have a continuous bed fall), should be filled up to 3 feet below ground level with quarry spauls and dry material from the waste-weir and other excavations. This filling should be of coarser particles at the base and of finer ones at the top, and should be lined at the sides and covered at the top by 1 foot of coarse sand and fine gravel. The whole should be finished off by soil, which, with advantage, might be carried a little above ground level, so as to mark the position of the trench and so as to prevent water from lodging above it.

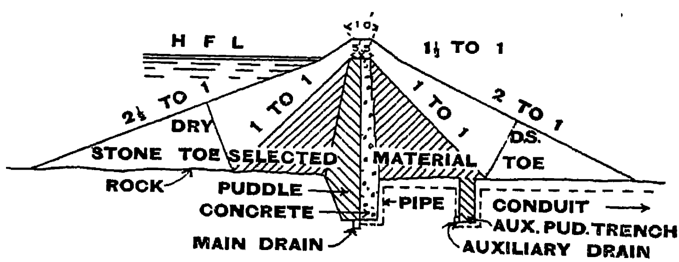
This drain will effectually drain the ground on the downstream side of the dam and for some distance downstream of itself; it will also intercept and pass off percolation rising from the puddle trench and prevent it from soddening the base of the dam. Further, it will have a decided effect in reducing the salt efflorescence which damages agricultural land

below some dams. This efflorescence is due to the upward passage of subsoil water, charged with dissolved salts, in excess of what the natural subsoil drainage can pass off. As this upward moisture is continually evaporated by the sun, the salts thus crystallise out on the surface and form an efflorescence, which is frequently fatal to plant growth.

The "foundation drains" (para. 112, p. 159) below the downstream part of the dam can be led by cross drains at intervals into this "downstream drain."

FIG. 11

SECTION



Mr. Kreuter[1] has proposed an "auxiliary drain," at the downstream toe of the central selected material, with an auxiliary puddle trench downstream of it to prevent water from passing beyond that (Fig. 11). He rightly observes that, not only is it necessary to prevent slipping from occurring within the mass of the dam, but also in the subsoils below it. It is to cut off percolation water from lubricating the subsoils, especially at their junction with the bed rock, that he has proposed this drain in addition to a puddle trench drain. The proposal is based on sound ideas, but,

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[1] "Minutes of Proceedings, Inst. C E.," Vol cxxxii., p. 263.

as the number of drains below the superstructure of the dam should be as few as possible, (as they cannot afterwards be attended to), it is questionable if this additional interior drain is necessary. The puddle trench drain and the downstream drain should drain the subsoil between them and downstream of the latter, but the auxiliary puddle trench must tend to keep moist the area between it and the main trench.

Mr. Kreuter also proposes to make the central core wall of two portions to secure perfect water-tightness—on the upstream side of plastic puddle and on the downstream side of rigid concrete. This seems to be a refinement which could not easily be carried out in practice.

108. "Rear Drains."—At all valley lines crossed by the dam, "rear drains" (Plate 4, Fig. 1, and Plate 5, Figs. 2 and 6) should be run at right angles to the centre line of the dam with as steep a bed slope as possible out to the surface of the ground. They will pass off the flow from the "downstream drain" and "puddle trench drain" to the natural drainage lines of the country. Where they are excavated in soil, they should be of the underground type (para. 109, p. 155).

The "main rear drain" in the river bed will generally be in rock, and should, anyhow, be in open excavation, so that its free discharge can be observed and maintained at all times. It should be protected from cross drainage, bringing in silt and *débris*, by means of good side embankments, and care should be taken that its outfall is not choked by material brought down by the erosion of the waste-weir channel. As it has to drain the highest part of the dam, the greatest care is necessary always to keep it clear.

The bed of the puddle trench at the river crossing will probably be near the surface, and, in such a case, it can be drained directly into the main rear drain. Where, however, it is at a low level and the natural fall of the rear drain is gentle, the former can still be drained into the latter by a short drain led upwards with a rapid slope from its bed and made to discharge above the full-supply level of the main rear drain, so that there may not be any back flow to the puddle trench drain from that drain. The sloping drain, by affording an easy exit to the puddle trench drainage, will prevent the base of the puddle from becoming sodden. This artifice of upwardly draining subsoil water (Elkington's system) has already been tried with success in agricultural practice in cases where a retentive soil on rising ground is underlain by an imprisoned, permeable, and water-bearing stratum. Its principal is similar to that of the artesian well.

To assist in maintaining the main rear drain clear, it is advisable to construct near its head a flushing pool, in which the drainage can be gradually stored and then passed out rapidly at intervals to scour out any silt that may have been deposited in the lower part of the drain. As the drainage water will itself be clear, this pool should not silt up, but probably it will have to be kept free of rank vegetation.

A small gauging weir notch with clear overfall should be placed above this pool, so that a daily record of the drainage discharge may be obtained. Similar gauges should also be fixed on other important drains.

As long as the drains run clear, it shows that no damage to the embankment is happening. Should, however, the flow be discoloured it is an unmistakable

sign that a leak is in process of formation, and, the longer such a flow continues, the greater will be the danger. The source of such excess percolation should at once be found and the defective part of the embankment should be cut out and remade, even if this involves the running of the storage to waste. Such cases are practically unknown in Indian dams, and will not occur if the construction has been sound.

A simple and convincing test of the efficiency of the drainage arrangements will be afforded by the condition of the ground at the rear of the dam. If that is free from excess moisture, it is certain that the drains are working properly. Swampy places, on the contrary, are sure indications that the subsoil percolation has not been properly intercepted by the drains, and the defective lengths should at once receive attention.

109. Open and Underground Drains.—Shallow drains in clayey soils are of little use for draining the rear of the dam, as they do not tap the deep-seated percolation water. This finally rises between them to the surface, and, being retained by it, soddens it and produces a marsh, in which grow in profusion reeds and water plants, which add a further obstacle to drainage. Such drains are also not useful for draining porous soils, which, before becoming charged with water, can themselves pass off most of the supersaturation, although they may become clogged by its silt thus concentrated. Deep drains, extending if possible to the bed-rock, are therefore required in both cases. If these drains are made as open trenches, they are liable to slips, to become choked with silt brought into them from the surface, and to be blocked by the growth of weeds, so that in a few years, if not

properly maintained, they are likely to lose most of their efficiency. The system of underground drains described in paragraph 107 (*b*), p. 149, is therefore recommended; such drains will continue to run clear, as their sides cannot fall in, surface silt cannot be brought into them, and weeds cannot grow in them. As they can generally be filled by refuse dry spoil, their cost beyond that of ordinary borrow pit excavation for the dam will not be very great, and is well worth being incurred for the advantages they secure.

110. The Self-drainage of Earthwork.—Ordinary Indian soils available for dam construction are not absolutely water-tight; in fact, it may be doubted if such earths exist anywhere. Even a very retentive soil will thus admit a certain amount of water into its mass, but, as the hydraulic gradient of this infiltering water falls the further it penetrates, if the section of the embankment is sufficiently wide, there will not be any surface percolation at its downstream limit, but the mass will be more and more charged with water the nearer the source of supply is approached. Retentive soils become greasy when wetted, and the worst varieties tend to become slushy when sodden. It is, therefore, most desirable to prevent the heart of an embankment from becoming saturated with water (para. 69A, p. 99), and for this reason measures should be adopted to make the earthwork able to drain itself of superfluous water, which would be dangerous should it lubricate the whole mass, or should it collect from a large area and find a defined line of escape. These measures consist in mixing with pure argillaceous soils a certain amount of shaly material, which gives the dam this power of self-drainage at the same time that it increases its frictional resistance to slipping.

If the dam is made wholly of dry, gritty material, that will, of course, allow the passage of water freely through the embankment, and such a soil should therefore not be used for its construction. There is, however, a mixture affording a happy mean between excessive retentiveness and excessive porosity, and one which, without inducing percolation at the upstream part of the dam, will enable the largest extent of the downstream part of the earthwork to be self-drained and thus maintained in the most stable condition. The mixture recommended, (para. 80, p. 118), is one part of pure black "cotton-soil" to one part of clean muram, or such other mixtures of existing soils as will result in the formation of a material with those proportions of clay and grit.

Theoretically, the parts of the section of a dam should become gradually more and more shaly and less and less clayey the further they are removed from the source of supply of the infiltering water. In practice, such a gradual change of material is difficult to carry out, and a uniform mixture is therefore desirable. If, however, there is an excess of gritty material available, it is best to utilise in it an increased proportion on the downstream side of the centre line of the dam. Every precaution should be taken on the upstream side of that line to make the earthwork as resistant as possible to infiltration; and on the downstream side to give it the property of self-drainage so that the percolation water, which penetrates the embankment, may quickly and harmlessly pass out of it.

VIII. The Foundations of the Dam.

111. Order of Suitability of Natural Foundations.—The best foundation for a dam is one of compact, unfissured rock, provided it is level, or does not slope steeply downstream, as then it would tend to cause a slip. A small downwards inclination of the surface upstream is not so great a defect, owing to the upstream slope of the dam being flatter and being supported by the water pressure. To prevent any slipping of the earthwork when the base is slightly sloping, the rock surface should be roughened by shallow trenches excavated in it parallel to the axis of the dam, and parallel core walls should be built at intervals projecting from it into the embankment (Plate 5, Fig. 2). If the surface of the rock is fissured it should be removed on the upstream side, and infiltration through lower fissures should be prevented by excavating cut-off trenches down to sound rock and filling them with fine concrete: small fissures should be grouted with cement.

The next best foundation is less compact rock, the unsound parts of which upstream of the puddle trench would have to be removed, but those which are downstream of it might be allowed to remain for the sake of the means of drainage which they afford. Following this is compact muram. These three classes of foundation have the great merit that they will not be sensibly compressed by the immense weight of the dam.

After these come, in the order named, mán (a hard clay soil), brown and red soils, and black "cotton-soil"; they all require special precautions for drainage, as they will yield under a heavy weight when sodden,

and if they are tilted to any great extent, may slip. Some engineers prefer a foundation of clay, so that the dam may unite perfectly with it, and consider that a rock foundation is liable to cause foundation leaks, but these can be prevented by a series of concrete trenches, whereas with clay it may not be possible to prevent deterioration of the foundations.

All soils which are light and powdery and wanting in cohesion, and all which under the action of water become slippery or slushy, are quite unsuitable either for the foundation or for the construction of a dam. Soils which contain carbonate of soda and deliquescent salts are particularly dangerous, owing to their power when wetted of dissolving the earth which contains them. If such soils exist for a moderate depth below the seat of the dam, they should be entirely excavated and thrown to spoil. If, however, they are deep and the expense of their complete removal is too great to be faced, the site will have to be rejected.

The essentials of a good foundation are that it should be of a compact nature, which when wetted will not yield, unevenly or extensively, nor slip under the weight of the dam. It is not necessary to have for an earthen dam the absolutely rigid base required for a masonry dam, but, the nearer the foundation of an earthen dam approaches this degree of excellence the better. The thorough drainage of the bed of an embankment is a matter of vital necessity. To ensure it, it is always well to select a site for the dam where the fall of the main drainage lines will facilitate the rapid passage off of all subsoil water. Fortunately, most dams are situated on ridge lines, which afford the required means of natural drainage owing to their elevation above the rest of the country.

112. "Foundation Trenches and Drains."—As stated in paragraph 106, p. 148, the maximum amount of percolation will naturally occur along the junction of the dam with its foundation; it is therefore necessary to take greater precautions here—firstly, to prevent infiltration as much as possible, and, secondly, to drain off harmlessly the water which has not been stopped.

Plate 5, Fig. 4, shows how this may be done. On the upstream side of the dam is a series of small puddle trenches, parallel to the main central one, of which one or more may be made of greater width and depth than the others to aid in preventing subsoil flow along a defined plane. Like the central puddle trench, they should be filled with as retentive material as can be found within half a mile of the site.

On the downstream side of the embankment is a similar series of trenches, but these are filled with porous material to form the "foundation drains," which will drain, uniformly and thoroughly, the whole of the overlying part of the dam. The porous material should be protected from silt infiltration by a cover of fine grit and should itself be arranged so as to have its coarser particles at the bottom and its finer ones at the top; it can be procured from suitable excavation spoil of a sound nature. It is not necessary that slab drains should be built at the bottom of these drains, as each of them will have to deal with but very little water. They should be excavated in discontinuous sections, say 300 feet long, falling uniformly to their downstream ends, where they should be connected by cross drains of similar section which should be carried at right angles out of the dam and continued well below the "surface drain" in covered slab vents leading to the "downstream drain" which will form

their outfall. To prevent the formation of long continuous drains under the dam, each section should be separated from its neighbour by an unexcavated strip, say, 5 feet wide, with its downstream side slightly sloped to the lower section. If it is considered desirable to maintain a constant watch over the flow of the cross drains, they may be made to discharge into the "surface drain," but the better arrangement for this will be to form small inspection chambers in them where they emerge from the dam.

It may be noted that even the most upstream of these "foundation drains" will be so far removed from the reservoir that they cannot possibly induce any leakage from it.

This system of drainage is far superior to one that is sometimes adopted in which there is only a series of drains at right angles to the centre line of the dam, corresponding to the cross drains described above. Such right-angled drains, occurring, as they will do, at intervals only, will drain the whole base irregularly, and will thus tend to produce unequal settlement and, possibly, a slip by concentrating the drainage along defined lines.

113. Benched Foundations of the Dam.—The whole seat of the dam outside the puddle trench should be stripped of unreliable soil and fissured rock where that is exposed on the upstream side, and should then be excavated into a series of large furrows parallel to the centre line of the dam, and having their troughs above the "foundation trenches and drains" (Plate 5, Fig. 4).

The object of these benches is to let the dam rest on a series of slightly inclined planes, which will tend to make its layers settle towards, and not away from, the

centre line, and will thus increase the stability of the whole mass. The downstream benches have the further advantage of leading all the percolation water from the overlying part of the dam uniformly to the "foundation drains," and of preventing any subsoil water from rising into the embankment. Benches are often made level; they then do not tilt the earthwork toward the centre line but rather have the effect of causing unequal settlement at the steps thus formed; nor, what is of greater importance, do they improve the drainage of the base of the dam. As the material excavated from the benches should be of good quality, it should be suitable for the formation of the neighbouring sections of the dam.

At the steep sides of the river gorge the benches should be formed in a different way, namely, at right angles to the axis of the dam and with a continuous slope falling towards the natural flank (para. 74 (*a*), p. 107). Here the object of the benching is to make the embankment settle tightly both on to its base and on to the sides of the gorge. To prevent the formation of direct leakage planes, unexcavated strips, say 5 feet wide, should be left on the upstream side of the centre line at intervals of, say, 50 feet, so as to close the furrows of the benches. On the upstream side of these strips small cross puddle trenches should be excavated parallel to them so as further to aid in cutting off leakage. On the downstream side of the centre line of the dam the "foundation drains" should be excavated in the troughs of the furrows, and each should be led out of the dam independently of the others.

This preparation of the surface, being slow work, is not suitable for the employment of famine labour, but,

being all in the open, can be undertaken by such labour if it is started when the numbers are few, or if certain selected gangs are drafted to the work after the numbers have become large. It could be done only at considerable expense in anticipation of famine work becoming necessary, as in that case the whole of the embankment would have to be raised a few feet above the prepared surface to protect it from injury.

114. Deep Black "Cotton-soil" Foundations.—Where these exist, it may be desirable to fill the downstream benches, and to make the embankment over them for a few feet in height, with double the ordinary proportion of gritty material, so as to give the superstructure a firm, insoluble bed over the whole area. This and the flattening of the slopes on both the upstream and downstream sides should sufficiently distribute the pressure of the dam on the subsoil (para. 74 (*b*), p. 109).

115. Sandy River Bed Foundations.—Where the river bed is of deep, compact sand with some clayey matter in it, it may occasionally be more economical to leave this unexcavated on the downstream side of the puddle trench, and to prevent its motion by a strong masonry toe-wall with drainage arrangements through it. The sand will thus act as a natural drain to the base of the dam. The part of the bed upstream of the puddle trench should, of course, be entirely cleared of sand, and, as a further precaution against direct infiltration, the puddle trench itself should be widened considerably. The chief danger from not excavating the downstream part will be the possibly unequal settlement of the two portions of the base of the dam, but this can practically be obviated by raising the dam slowly, and by giving it a year in which to settle after

completion before the reservoir is allowed to fill. Should the expense of these precautions approach the cost of the entire replacement of the sand by embankment, it is obvious that it will be better to effect that replacement rather than to economise a little by retaining the natural sand in the downstream part of the river bed.

IX. The Construction of the Dam Superstructure.

116. General Formation.—The method of constructing the dam is :—first to wet slightly the layer last completed ; on the moistened surface to spread and afterwards mix the material of the new layer ; and then to consolidate it. On the completion of a layer, the process is to be similarly repeated for the next one, and so on. These operations are described in detail below.

117. Watering.—The sole use of water is to unite the constituent layers of the dam into one solid, unfissured mass. The quantity of water used should be restricted so as to be sufficient only to form a thin film of moistened material on top of the completed layer, into which the dry material of the layer under construction may be forced during the process of its consolidation. Any excess of water beyond this will make the layer too moist and will expand its constituent particles, thus rendering it less compact as a whole (para. 63, p. 92). Should a great excess of water be used, it will form slush, which will be evidenced by the upper layer moving before the roller with a wavy motion and cracking. Such slushy parts should at once be cut out, the excavated material should be put on one side, and allowed to dry before it is again

used, and fresh dry material should be carefully consolidated in its place.

Where the soil used in construction is very dry, it is a good arrangement to wet overnight the borrow pits from which it is obtained and then it will be found on the following day suitably damp for excavation and use.

After a few feet in height of the embankment have been constructed, it is advisable by means of a hose, to pour water over the slopes for several days until they become so consolidated by the washing in of the fine particles of the soil that the water no longer penetrates, but at once runs off them. The slopes will then be found to be extremely compact for a depth of 3 or 4 feet, and each to have so hard a surface that the one on the downstream side will not be guttered by rainfall, while that on the upstream side will form a sound bed for the pitching.

118. Spreading and Mixing.—The clayey material should first be evenly deposited on the finished layer and should then be evenly covered with the shaly material; the thicknesses of the two should be regulated by experiment so as to produce the finished mixture with the proper proportions of the two constituents. All clods should be broken up by hoes or wooden rammers. The two materials should then be thoroughly incorporated together; this can best be done by hoeing them by hand, but, as this is expensive, they may be mixed by harrows with long obliquely projecting wooden teeth, or by light inverting steel ploughs.

The greatest care must be taken to produce a uniform mixture and to avoid stratification, which not only might lead to infiltration along the porous

planes, but also to slipping along the clayey planes, should such be formed by imperfect mixing. The shaly material is required to give the earthwork frictional stability, to unite the different layers completely and intimately with each other, and to permit of the whole mass obtaining the power of self-drainage. The clayey material is required to give the whole mass cohesive stability and to make it resistant to water infiltration. The mixture having been thoroughly made, the whole dam will settle and act uniformly under all conditions, and unequal stresses will thus be avoided.

An instance showing the danger of forming a dam with stratified layers separated by dry porous planes is afforded by the failure of the Huli Ela reservoir in the Southern Province of Ceylon. That work was constructed about 1870 and had a full-supply depth of only thirty feet. About 1912 it was breached, and the cause of the breach was clearly seen to have been seepage between the dry planes separating the layers, as the earthwork was otherwise well consolidated. Another lesson to be learnt from this accident is that many years may elapse before failure occurs to a dam faultily constructed (para. 143, p. 195).

As noted in paragraph 81, p. 120, the casings will be formed of material richer in grit; being formed and consolidated at the same time with the hearting, they will be perfectly united with it.

119. Consolidation.—To render the whole of the earthwork perfectly uniform, it is essential that during construction it should be artificially consolidated equally throughout the section. Thus, during the final consolidation due to the settlement of the earthwork the material will be perfectly regular, the original

disposition of the layers will be maintained unchanged, and the formation of leakage and slipping planes will be prevented. Should such planes be produced, it will not be possible at once to remove them, as their existence will not be known until they cause signs of failure.

To avoid an excessive and sudden amount of settlement, which may not act uniformly throughout the mass, the embankment should not be raised more than 30 feet in height in any one season. During the subsequent monsoon, at least half of the total final settlement will have occurred, and the substructure will thus be compact enough to permit of the safe raising of the superstructure upon it.

Entire dependence must not, however, be placed upon this self-consolidation of the dam, for if artificial consolidation was not originally effected, the settlement of the mass will be much greater and much less uniform, owing to the varying heights of the embankment in its longitudinal and cross-sections, and internal stresses may be set up. Moreover, an originally loose structure will at first be subject to great infiltration from rain and from the water of the reservoir, which will keep it green for a long time, and this will tend to cause still further unequal settlement and less resistance to further infiltration (para. 69A, p. 98).

After the layer under construction has been mixed and levelled, a light roller, weighing, say a quarter of a ton per foot run, should be passed over it to produce an even surface, on which a heavier roller, say 4 feet in diameter and in length, and weighing three-quarters of a ton per foot run, can work easily. For small dams this latter roller will suffice to give enough consolida-

tion, but for large ones it should be followed by a steam roller weighing eight to ten tons. A roller worked by animal power should consolidate in a day 20,000 square feet, and a steam roller, 50,000 square feet of surface. A steam roller consolidates the dam more thoroughly than lighter rollers worked by animal power, and thus reduces the amount of subsequent settlement and consequent tendency to produce settlement cracks; it occupies less space on the dam than an equivalent number of lighter rollers; and its use obviates the employment of so many animals which may be difficult to obtain in ordinary seasons and may not be available in famine years, and, anyhow, crowd the work. Rough tests of the sufficiency of the rolling are that the roller will not further compress it, and that loaded carts passing over the surface will leave but faint rut marks on it. All rollers should be provided with automatic scrapers to prevent them from lifting the earth in caked masses, and for this reason cast-iron, and not stone, rollers should be used.

In America grooved rollers have been used so as to concentrate their weight on the area of their projecting rings; this will, however, render the compression uneven, and their chief advantage appears to be that they will knead the layers together and prevent stratification. For the edges of the earthwork it is advisable to have split rollers, fixed on a common axle, and projecting beyond the track of the animals drawing them. It is best to have all ordinary rollers split so as to enable them to turn more easily. At the top of the dam, where the lessened width will not allow animals to work, the consolidation should be effected by light stone or iron rollers weighing

about a quarter of a ton, and worked by men. One of these can subsequently be used for the maintenance of the top of the dam.

In confined situations, such as at the junctions of two lengths of the dam, the ordinary roller cannot work, and resort will have to be had to hand ramming. The men should work in unison and be spaced apart a distance equal to a little more than twice the width of the bottom of the rammers, in order that they may strike the embankment with them alternately on one side and then on the other, so as completely to cover the area, and so as to knead the earth together as they progress slowly forward. As ramming does not compress earthwork as much as does rolling, the places which are rammed are ones where the final settlement will be greater than that of those which are rolled, and, at the junction of the two, there will be a plane of unequal settlement. To avoid as many such junctions as possible, long, continuous lengths of embankment should be constructed at the same time, and the junctions should be made in steps of low height, breaking joint well with each other (Fig. 12, para. 123, p. 171).

Surface cracks and subsidences of the slopes may possibly occur during the monsoon when the dam is new. The cracks should be completely filled with a mixture of grit and soil well worked in by means of chisel-pointed poles 8 or 10 feet long. The surface, at places of settlement, should be slightly excavated and then restored to the proper slope by adding fresh material, which should be carefully consolidated by rammers and hand beaters, so as immediately to shed the rainfall.

120. Thickness of Layers.—The thickness of the

layers has to be regulated so that the maximum consolidation is attained at the base of the dam, where the water pressure will be the greatest, and so that less and less consolidation is effected as the work rises and the water pressure decreases. It would cost too much in time and money to consolidate the whole dam throughout to the maximum extent, and this amount of consolidation is not necessitated elsewhere than at the base by the conditions which have there to be met.

To obtain this gradually increasing consolidation the first 30 feet of the dam measured from the top should be constructed in layers 5 inches thick when rolled by ordinary rollers; the next 30 feet in 4-inch layers; and the remainder of the dam and the river crossing in 3-inch layers. Where steam rollers are used, these thicknesses of the layers may be increased by 50 per cent. The loose layers will roll down to about two-thirds of their original thickness. Owing to the great pressure of the dam, the shrinkage of the natural earth excavated for its construction will be about 6 per cent., *i.e.*, about 106 cubic feet of excavation will be required for 100 cubic feet of embankment, and this should be allowed for either in the borrow pit measurements or in the excavation rate.

121. Slopes of Layers.—To counteract the natural tendency of earthwork under pressure to slip outwards, the layers should be formed with slopes inclined downwards to the centre line of the dam (Plate 5, Fig. 4). The slope from the downstream edge should be made as steep as watering and consolidation will permit, say 1 in 10. The central eighth of the layer should be made level; the upstream edge should be

kept at the same level as that of the downstream edge, and the slope from it adjusted accordingly. The resultant inwardly-slipping tendency of the downstream and upstream parts, having masses of different size and being constructed with slopes of inversely proportioned inclination, will thus be approximately balanced (end of para. 147, p. 198). Care must be taken during construction that the water used for moistening the dam remains uniformly distributed over the surfaces of the inclined layers and does not run down them so as to collect at their bases.

During the self-consolidation of the dam, as the interior portions are of greater vertical height than the exterior ones, and are thus subjected to greater pressure, even layers which were originally level must tend to settle down towards the centre, but to so small an extent that this natural settlement cannot by itself add appreciably to the frictional stability of the embankment.

122. Uniform Construction necessary.—To avoid the formation of slipping planes, the whole width of the section must be formed and consolidated at the same time. Moreover, as it is not possible thoroughly to consolidate the extreme edges, the width of the dam should originally be constructed 18 inches or 2 feet on each side in excess of the final width. After the dam has been raised a few feet, this extra width should be dressed off and the material from it used for making up the higher part. For the same reason no patchings-on of the casings, etc., should be allowed, and all works roads leading up the dam should be formed with it outside of its designed section and should be removed when no longer required (para. 76, p. 112).

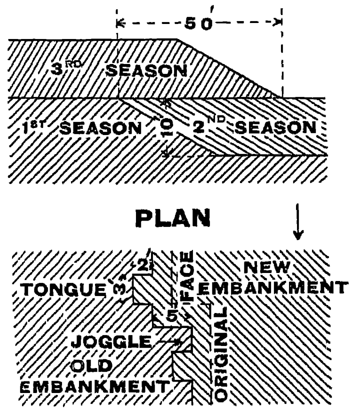
Every precaution must be taken to construct the dam as carefully as possible so that it may settle uniformly under all conditions.

123. Junctions of Earthwork.—Where junctions of earthwork are unavoidable, they should be most carefully constructed.

(*a*) *Cross-sectional Junctions.*—These should be made as shown in Fig. 12.

FIG. 12

SECTION



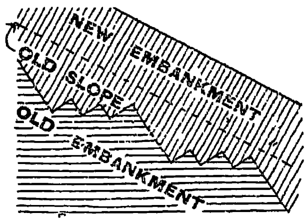
All the loose surface earth of the old slope should be entirely removed, and, as shown in plan, the junction should be made with slip tongues and joggles inclined up the face of the slope to allow the new embankment to settle tightly on to the old one. Each junction should not be made more than 10 feet in height, and, where a greater height has to be dealt with, it should be broken up into steps separated by horizontal breaks of at least 50 feet. All the junctions together should not be carried up more than 20 feet in height in one season, and the work of the different seasons should break joint as shown in the sectional elevation.

(*b*) *Longitudinal Junctions.*—These junctions, or patchings, should be made as shown in Fig. 13, by benchings of irregular size, thus designed, so as to prevent the formation of a slipping plane; the new earthwork should, moreover, be constructed in layers sloping steeply on to the old surface. Transverse filtration will not be induced by such a junction,

but, as this form of construction has somewhat of a tendency to slip, it should not be made if that can be avoided. Not more than 20 feet vertically should be carried out in one season.

FIG. 13

SECTION



(*c*) *Additions to Height.*—Whenever new earthwork has to be added to the top of old earthwork, *e.g.*, when an old dam with a large top-width has to be raised, in addition to removing the old surface, one or more key trenches, say 4 feet wide at the base by 3 feet deep, should be excavated in the old work with slightly sloping sides and parallel to its centre line. These should be filled with the most retentive material, thoroughly rammed, before the main part of the new earthwork is commenced.

124. Finishing-off the Dam.—At the end of each season's work on a section of the dam, the outer edges should be left a little higher than the centre, so that the rainfall on this wide top area may not flow down the slopes and gutter them, but, on the contrary, may soak gradually into the earthwork and help it to settle. Excess water should not be allowed to remain on any part of the top, but should be drained off by gutters designed so as not to scour the slopes at their outfall.

The top foot of the completed dam should be made of porous material for the same reason, and the top surface should be given a slight fall, of say 1 inch, towards the reservoir; the drainage from it will thus have only a short course to the pitching and will

therefore not gutter the upstream slope. During construction and maintenance care must be taken that the drainage descends evenly down the slopes, and that it is not allowed to concentrate down any defined line, as this may result in the formation of deep rain scores. The top of the dam should be finished off with half an inch of coarse sand well rolled in.

To protect the downstream slope it should be sown with grass, or turfed, so as to enable it to resist the guttering action of the drainage of the rainfall passing down it.

A good fence or hedge should be made round the dam to protect the embankment from cattle, etc.; on the upstream side this should extend down to full-supply level, and on the downstream side it should be carried along, and downstream of the "downstream drain" (para. 107 (*b*), p. 149). During construction a good works road should be made along the downstream toe of the dam, and should be carried across all stream beds on drained berms and thereafter should be maintained to facilitate inspection.

125. Observations during Construction.—To test the nature of the construction, at the close of each week small trial pits, say 2 feet square, should be excavated at intervals through the week's work. The earthwork, if it has been properly constructed, should be found compact, uniform in composition, free from distinct stratification, and only slightly moist. To test its impermeability, the pits may be filled with water and the time this takes to disappear may be noted. Any defects thus brought to notice should be remedied during the subsequent work.

During the rains theodolite and level observations of stout pegs driven 4 feet deep, projecting 1 foot above

the surface and spaced at regular intervals on cross-sections and on fixed lines ranged parallel to the centre line on all high parts of the dam, should be made and continued until the close of the monsoon succeeding the completion of the work, and a continuous record of them should be maintained. Thereafter, if great settlement or distortion has not been observed, it will be sufficient to keep an annual tabulated record of the level of bench-mark stones firmly bedded 2 feet deep at the end of every chain on the top of the dam along its centre line; these stones should be engraved with the chainage for easy identification and so as to form permanent distance marks.

Gauging notches should be fixed at the outfalls of all drains and a continuous tabulated record should be kept of their discharges.

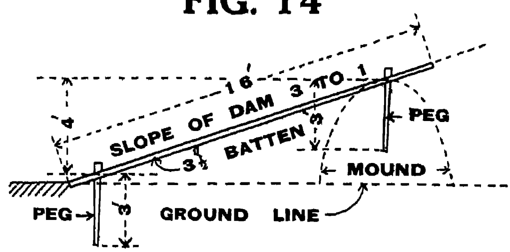
126. General Remarks.—(*a*) *Setting out the Centre Line.*—Dams should be set out in as long straight lines as possible, so that the appearance of the finished work may be in keeping with its scale. Following minutely the irregularities of the ground will not as a rule effect much saving compared with a carefully selected equalising line. These straight lines should be united by short curves, as long ones involve trouble in setting out, and there is no engineering necessity for the latter as exists in the case of a canal, road, or railway.

Reference setting-out pillars should be built at the extremities of all straight lines, and, if practicable, clear of the dam, so that they may be of use during the whole of its construction and also after its completion; from them the intersection points of the different lines can always be set out (Plate 4, Fig. 1).

Setting out Templates.—The toe of the dam should

be set out as sketched in Fig. 14, a small mound of the casing material being first formed to take the upper vertical peg. The template should be made by a good 3-inch by ½-inch sawn teak batten about 16 feet long, as strings and bamboos are not sufficiently accurate for good work. These templates should be fixed 50 feet apart, opposite to and intermediate with, the regular chainage marks. When the embankment has been raised above the top of the template, the slope may be continued roughly by eye, more especially since it has afterwards to be dressed off (para. 122, p. 170). After the dam has been carried up some feet above a tem plate, another template can be fixed in continuation of it and the earthwork dressed off to the lower one.

FIG. 14



In the case of cuttings, excavation templates, about 1 foot wide, should be neatly dug to the finished slope, at intervals of 50 feet or so, and the earth lying between them should be dressed to them.

(c) *Minor Arrangements.*—In Appendix 22, p. 448, are given numerous notes of these arrangements. These notes were originally issued as works orders, and will, it is hoped, prove useful.

(d) *Programme of Work.*—Before the commencement

of each season's work should be made out a programme of the quantities to be executed and the levels to be reached, month by month. This should allow liberally for all probable delays caused by holidays, agricultural operations taking labourers away, etc., etc. In actual execution every endeavour should be exerted to get in advance of the programme, so that, at the end of the season, the progress effected may be greater, rather than less, than that originally scheduled. Such a programme is particularly necessary for the closure of a high dam (para. 135, p. 187).

The programme should be made out as a tabular statement for quantities and as a progress section for levels.

(*e*) *Method of Executing Work.*—It will generally be found best to carry out a large dam by the petty contract system under departmental management, as a large contract is not, as a rule, advisable for this class of work, where the most careful construction is necessary, as well as rapid progress. Practically all the material being close at hand, and easily worked, and nearly all the labour being unskilled, there is not any necessity for the employment of a large contractor, whose principal use is that he has special appliances and skilled labourers at his command.

X. Compound Dams with Drystone Toes.

127. Objections to High Earthen Dams.—As mentioned in paragraph 66, p. 95, although French engineers consider 60 feet to be the safe maximum height for an embankment, English engineers do not agree with this opinion, as they have constructed many

much higher dams.[1] The latter generally hold that the proper section for high dams is one in which the slopes are made flatter towards the base than they are near the top. This corresponds with the "empirical section" (para. 62, p. 91, and Plate 5, Fig. 2), but that is not recommended as a perfect form for a high earthen dam. When a greater height than 60 feet is contemplated, it must be recognised that particular care must be taken both with the design and with the construction. Earthwork being viscous, a small disturbance at one point is likely to extend and to cause in time a larger one, and it is therefore necessary to guard against this. The higher the dam and the more that height differs at contiguous sections, the greater the chance of failure, as then there will be increased and varying settlement. Unless this greater and irregular settlement is provided against by careful and thorough consolidation, the more will it differ in amount at such sections, and larger internal stresses will thus be caused at them.

Now, the highest sections of dams are usually in the worst situations for stability. The most economical line for the whole dam is generally along ridges which terminate abruptly at the river, there forming a gorge through which the stream passes. At this point the steep end-slopes of the ground tend to make the

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[1] The following are the maximum heights of certain high earthen dams:—

1. Druid Lake Dam, Baltimore, 119 feet.
2. Temescal Dam, California, 105 feet (built in 1868, largely hydraulic-fill).
3. San Leandro Dam, California, 120 feet (built in 1874–5, one-third hydraulic-fill).
4. Croton Dam, New York, 120 feet (proposed, *vide* paragraph 58).
5. Wághád Tank, Bombay, 95 feet (original design).
6. Mhasvad Tank, Bombay, 80 feet (*vide* Appendix 1, No. 14).

No. 1, *vide Engineering News*, Vol. xlvii., No. 8, p. 152, February 20th, 1902. Nos. 2 and 3, *ibid.*, No. 10, pp. 194 and 195, March 6th, 1902 (See also the end of paragraph 66, p. 95.)

embankment slip off, and, if the bed of the gorge has a longitudinal slope falling downstream, which it frequently will have at such a site, this will accentuate the tendency to slipping. The upstream portion will be held up by the water pressure of the reservoir and by its greater mass, but the downstream portion will be dependent upon itself alone for support against the resultant force which will be directed towards it. The tendency will therefore be for the dam to slip downstream, and the slips which have occurred in a few Bombay dams have nearly all been in this direction, and very few slips of the upstream portion have taken place there. Of course, if the upstream slope be steepened, so that, with the reduced co-efficients of cohesion and adhesion produced by water infiltration, it is relatively weaker than the downstream one, it will be the first to give (see also para. 57, p. 81). The result of experience is that the usual 3 to 1 water-slope, although it is charged with water, is more stable than the 2 to 1 downstream one which is not subjected to the direct influence of the reservoir storage. In India, however, this latter slope, as described in paragraph 80, p. 118, is exposed to varying conditions which affect its stability, and, in the case of high dams, it appears necessary that there should be less difference in its angle of slope to enable it to have as much slip-resisting power as the upstream slope. It is for this reason that the "empirical section" shown on Plate 5, Fig. 2, provides for a relatively greater increase beyond the usual section to the downstream than to the upstream slope.

128. The Drystone Toes of the "Compound Dam."—If the gorge embankment can have its slopes confined by strong toes of some non-viscous material up to the

base level of the flanks (beyond which the dam is of a less abruptly varying height and where the difference of settlement of neighbouring sections will relatively be very small), the enclosed earthwork will be prevented from moving in any direction. After the base of a gorge dam has been allowed to attain practically final consolidation by settlement, the upper part can be safely raised on this reliable foundation to the ordinary section of the flank embankment. This constitutes the principle of what is here termed the "compound dam" (Plate 5, Fig. 2). The toes, if formed of trap dry-stone, which has a specific gravity of 2·50 and a co-efficient of friction of 0·71, compared with 1·60 and 0·50, the relative figures for moist clay, will have a frictional resistance about $2\frac{1}{4}$ times that of the latter. To give them cohesion and to prevent on the upstream face the inflow of water, which would lessen the effective weight, the drystone[1] should be packed solid with clayey muram, which will also considerably increase its frictional resistance. The stone would, of course, be the roughest of sound rubble, and probably much of it could be obtained from the waste weir and other excavations. It should be laid with its beds normal to the slope to increase the frictional resistance.

It might be said that the upstream toe is not so necessary as the downstream one, owing, as noted above, to the greater stability of its flatter slope, but as it will always be under water after the filling of the reservoir, and cannot easily be added to, it is advisable to construct this toe as a matter of pre-

[1] The term "drystone" has been applied to this form of toe to distinguish it from a construction of masonry with mortar. "Packed stone" would describe it more clearly.

caution. Moreover, its downstream concrete wall, as explained in paragraph 136, p. 188, will be of great use in the closure of the gorge embankment.

It might also be urged that the drystone toes might be replaced by masonry retaining walls, but the proper design for these with so great a surcharge would be somewhat doubtful, and sufficiently massive walls would probably exceed the proposed toes both in cost and in time for execution. It would certainly be hazardous to construct retaining walls of the same rough class of work, and the material saved by their use would not compensate for the loss of the extremely stable form of the proposed toes, which, in addition to their greater weight, also receive the normal pressure of the dam and distribute it directly over a large, cohering base.

Finally, it might be proposed to substitute for the drystone toes massive earthen berms forming a platform raised to the top of the flanks of the gorge. Such berms to give equal foundation support to the superstructure of the dam, as the drystone toes, would have to exceed them greatly in quantity (para. 128, p. 179). As they could not be carried out safely in stages during the closure so as to reduce the amount of each season's work, it would not be practicable to construct them for a high dam during the short season available (paras. 135 and 140, pp. 187, 192).

The type of dam with drystone toes is therefore recommended as the one most suitable for embankments exceeding 75 feet in height, and by its adoption dams may safely be constructed up to 125 feet in height.

129. The Construction of the Drystone Toe.—The toe could be constructed by unskilled labour in the

following way:—On the completed layer of stone would be spread a thin layer of clayey muram, and this would be thoroughly worked in to fill all the interstices immediately below it. The stones in the next layer would be laid by hand to interlock and break joint with the underlying stones, and would be malleted firmly on to them. Large interstices would be filled with single stones driven home, and small ones with clayey muram thoroughly worked in by short bars. Finally, the layer would be completed by levelling it up with the muram so as to leave only the tops of the stones projecting in order to bond with the bases of those of the next layer. This construction should not cost more than three times the rate of the ordinary dam embankment. As all the material could be stacked beforehand close to the site, the drystone toe should be constructed at nearly the same pace as the earthwork, which is a matter of great importance when the closure of the dam is taken into consideration.

To give greater frictional resistance and to prevent the infiltration of water, rubble masonry, or concrete, toes and core walls should be built into the foundation, so as to project into and key with the superstructure. The water face of the upstream toe should be laid in masonry, or covered with concrete, to prevent water from entering and buoying it up; and its downstream face could be formed as a battering concrete wall to enable the earth pressure of the dam to be evenly distributed over it.

130. The Advantages of the "Compound Dam."—In paragraph 53, p. 76, the advantages of earthen embankments have been described. In addition to those possessed by an ordinary earthen dam, the compound dam has the following:—

(1) It protects the base of the rear slope from being guttered by rain;

(2) It can be constructed to a considerably greater height;

(3) It can be raised subsequently by a largely increased amount;

(4) It offers great facilities for the safe closure of the work.

The compound dam is more expensive than the ordinary earthen dam, but is cheaper than a masonry dam or than a composite dam, which consists of separate lengths of masonry and earthwork in continuation of each other. It has the advantage over a composite dam that its hearting is continuous and uniform throughout the whole embankment, and thus there is not any tendency to the formation of leakage planes through it.

XI. The Closure of the Dam.

131. The Importance of the Closure.—The closure of a high earthen dam gives the Indian engineer the greatest anxiety, as a passage for an immense quantity of flood water has to be provided temporarily during the monsoon through the embankment until the beginning of the last season, when the last gap has to be closed within seven months. No considerable amount of work can be done after the commencement of the monsoon, as the reservoir is then liable to be filled within a few days. During all the working period of the final closure a very large body of labourers must be kept constantly at work. The fear of an outbreak of cholera, which would effectually stop all progress, is, also, ever present to the engineer.

132. The Ordinary Method of Closure.—The ordinary system of the closure of a dam is shown in Fig. 15.

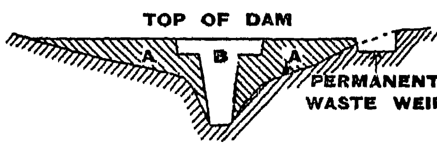
The flanks A A are first constructed, leaving the narrowest possible gap in the river bed for the passage of the monsoon discharge as by-wash channels are not practicable on account of their cost. The central part B has then to be completed during the last season of work. The objections to this method are:—

(*a*) The junction between A and B is too high. A, having been made at least one season before B, and having been allowed to settle in the open, at the time of junction has attained most of its final consolidation, whereas B is quite green. The two may not properly unite, and there is always a risk of a leak forming at their junction.

(*b*) B, while green, may be at once subjected to the infiltration due to a full reservoir; the water thus entering it prevents it from attaining the density of A for many years.

FIG. 15

USUAL CLOSURE



(*c*) B, having to be carried up the full height in one season, cannot be allowed to consolidate gradually by its own weight; it is, therefore, liable to internal stresses and distortions due to unequal settlement, which will chiefly be caused by the varying effect on the cross-section of the infiltration of water from the reservoir during the first few months.

(*d*) B, having to be completed in one season, the work is, to an undesirable extent, in the hands of the labourers, and strikes and higher rates may occur.

Moreover, there is the danger of cholera interrupting its progress.

This method of closure can, however, be adopted without incurring much risk for dams less than 30 feet high, but it does not seem proper to employ it for ones of greater height.

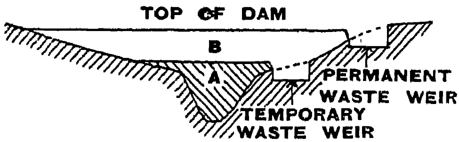
133. The Revised Method of Closure.—The revised system of closure is shown in Fig. 16.

It consists in forming the dam in continuous level stages, commencing with the one at its lowest base, A, and working gradually up to that forming its final crest, B. To provide for the passage of flood water during its construction, a temporary waste-weir is made at a suitable site in the longitudinal section, so that it can discharge the maximum flood with safety to the whole work. This system of closure is the only safe one to adopt for dams of greater height than 30 feet. Its advantages are :—

(*a*) The junction between A and B is very much less in height than in the previous case. The height of A at the junction can, at little further expense, be reduced so as to form a longitudinal step in the dam, by making its top there as a crest enclosure dam of small section; this can be removed, partly or wholly, when completing the dam at the time of final closure. With this arrangement the top of the dam, as finished for the preliminary closure, will be stepped in profile, both in cross and in longitudinal section, the crest dam

FIG. 16

PROPOSED CLOSURE



being raised above the rest of the unfinished part of the main dam inside of it.

(*b*) A is at first subjected to infiltration only from a shallow reservoir; all the upper part of the dam, with the exception of the comparatively low part closing the temporary waste-weir, can be allowed to consolidate before water touches it.

(*c*) The bulk of the dam, and, in particular, its highest sections, can be carried up slowly and evenly and allowed to self-consolidate. Only the small portion closing the temporary waste-weir will have to be done in one season.

(*d*) The work can be executed without fear of strikes, as the final closure of B involves a comparatively small quantity of embankment.

(*e*) The construction of A will lead to the formation of a small reservoir which will be most useful for works purposes, and will save charges for water.

(*f*) The temporary waste-weir will often provide a level site for the location of the outlet, away from the gorge embankment, and below the general bed of the dam.

134. The Temporary Waste-Weir.—As the temporary waste-weir is an essential to the closure of a high embankment, when selecting a site for such a dam it should be seen that the site offers facilities for the construction of this means of passing off floods safely during the progress of the work. In very high dams it may be necessary to extend the system by having one or more temporary waste-weirs at different levels, but the principle of execution will be the same as that to be followed in the case of a single one. The expense of forming more than one temporary waste-weir may be considerable, but, if an outlet of the type of the headwall in the centre line of the dam (para. 205,

p. 298) is adopted and is located at the site of this weir, it will generally be possible to utilise it as the sole temporary waste-weir until the completion of the construction of the dam. The method to be employed in this case is shown in Plates 4 and 9. During the first season the temporary waste-weir is left as an open channel. Afterwards, the lower part of the outlet headwall is raised to a certain height, calculated in accordance with the requirements of the construction of the dam, so as to form a clear over-fall weir, and its sluice-ways are left open to increase its discharging capacity.

The site for the permanent waste-weir should be kept open until the last (*i.e.*, the weir wall should not until then be built across it), so as to permit of the discharge of the flood waters at the lowest level.

The best natural site for a temporary waste-weir is at a depressed saddle, as then the floods from it will be confined in a natural channel, and will not tend to injure the main work. Where such a natural site does not exist, an artificial one should be made by excavating a channel so as to keep the tail flood below ground level. In either case it is advantageous that the bed of the temporary tail channel should be of rock, but, as this weir will come into action only for a year or two, it is not essential that the rock should be as sound as is desirable for the permanent weir. Where sufficiently sound rock does not exist at the proper level for the bed of the temporary tail channel, the erosion of that bed, under and near the base of the dam, can be prevented by constructing a masonry curtain wall a little below the line of the toe of the dam, and also, if necessary, by building water-cushion walls (para. 178, p. 245) downstream of this.

The discharging capacity of a temporary waste-weir must be equal to the maximum flood which can be expected from the catchment area, and, indeed, it will be better, as a precautionary measure, to make a somewhat greater provision than this, since, during the time this weir will be in operation, the whole of the works will be in an unfinished state. Account may be taken of the flood-absorptive capacity of the reservoir (para. 183, p. 252), but it must be remembered that this will always be small at the low level of the sill of the temporary waste-weir unless the maximum flood depth to be allowed over that is considerable.

135. The Closure of a Large Dam.—In a purely earthen dam the closure of the river gorge should be effected by carrying the full section right across the gap up to the required height above the temporary waste-weir. If, on the one hand, a reduced section of the embankment is first constructed on the upstream side, and the downstream part is subsequently added, this latter will have a tendency to slip off during its settlement and under the immense pressure of this, the highest part of the dam. If, on the other hand, the width of the gorge is first reduced by patching on flank embankments, during settlement there will be a tendency to produce cracks and leakage planes at right angles to the section, and these will induce infiltration of water from the reservoir into the heart of the dam. By the system of forming the base of the gorge embankment by massive drystone toes (para. 128, p. 178), both these methods of reducing the amount of work involved by the closure may, however, be adopted with perfect confidence, as these toes will give a large amount of support and

obviate slipping, and their construction will prevent the formation of a direct leak through the dam.

As an example, the method of closure proposed for the large Máládevi dam project is illustrated in Plate 4, and Plate 5, Figs. 2 and 5, and may thus be described :

136. The Preliminary Closure.—For this the whole section of the upstream drystone toe of the gorge embankment is to be raised to R.L. 124 00. A central gap, 300 feet in crest-width, is to be left in it, and the flanks of the gap are to be sloped at an inclination of $1\frac{1}{2}$ to 1 up to R.L. 166 00. The whole of the downstream face of the toe is to be protected by the "concrete batter wall" from the action of the river floods. The earthwork of the dam hearting at the flanks is to be set well back in low steps, out of reach of the direct rush of the floods, and its face is to be protected from erosion by heavy pitching, which will subsequently be removed and utilised for the permanent facing of the water-slope of the dam. The high-flood discharge is calculated to rise to about 20 feet on the sill of the gap, *i.e.*, to R.L. 144·00.

The raising of the flanks with full section to R.L. 166·00 provides an ample margin for safety, and this height might be reduced without risk if time did not allow for so much construction being carried out.

The "central wall" of the dam is to be completed with its overfall crest at R.L. 112·00. The width of its crest is to be contracted to 250 feet so that the flood water may be heaped up higher in order to secure a greater depth of water-cushion above it. The actual top of this wall, which is designed to key into the dam earthwork and from its small section might be washed away by the floods, is left to be constructed afterwards during the first complete closure.

The flanks of the downstream toe and its concrete core walls are to be constructed so as to confine the river floods. Its concrete toe-wall is to be carried right across the river bed to form a water cushion. The central gap in it is to be widened to 325 feet in order to reduce the flood-level, as the toe-wall will give ample water-cushion depth for the small overfall at the " central wall."

137. The First Closure.—For this the temporary waste-weir channel at the outlet site has to be made ready (Plate 9, Figs. 1 to 4). It is to be excavated with a deep central channel, 50 feet wide, carried down to the outlet sill, R.L. 135·00, so as to tap the floods early and to bring the reservoir flood-absorptive capacity (para. 184, p. 253) sooner into play, and so as to form a channel to serve for the discharge of the permanent outlet. At each side is to be a side channel, 75 feet wide and with its bed 10 feet higher, which is to provide for the balance discharging power required.

Appendix 12, p. 370, gives a calculation of the flood which can be disposed of by the combined effect of the channel discharge and the absorption in the reservoir during the rise of the water surface. Starting with a discharge of 5,300 cusecs (which is equal to a run-off of 1·28 inches a day from the catchment, and is a fair small flood), it will be seen that in nine hours they dispose of a total flood equivalent to a run-off of 4·75 inches. Of this, 2·99 inches are passed off by the channel, and the balance, 1·76 inches, is absorbed by the reservoir. When high-flood level is reached, the discharge of the channel will be at the rate of 0·546 inch an hour run-off, and assuming that the reservoir remains at this level, the total run-off

during the day will be 12·94 inches. These runs-off are far in excess of what may be expected from the catchment, and the maximum high-flood level possible may therefore be safely taken as R.L. 160·00.

For this closure the upstream drystone toe of the gorge embankment has to be completed across the gap left for the preliminary closure, and the "minimum section for closure" (Plate 5, Fig. 2) of the earthwork of the dam has to be raised behind it. This reduced section is designed so that the quantity of work to be done may be practicable within the time available. Should progress be more rapid, as is desirable, then the downstream drystone toe and the rest of the earthwork between it and the "minimum section for closure" should be raised simultaneously. Anyhow, it will be advisable to complete the whole section of the base of the dam to R.L. 166·00 and allow it to settle for at least a whole monsoon before the upper part is raised upon it.

138. The Second Closure.—For this the outlet head-wall is to be raised to R.L. 147·00 (as shown in Plate 9, Fig. 1, by plain dotted lines), and the under-sluice vents are to be left fully open in order to increase the discharging power. The upper plain dotted line indicates the high-flood level R.L. 169·00 as calculated in Appendix 13, p. 372. As the temporary crest will be constructed in steps, for the sake of raising the superstructure safely upon it, it will give a larger high-flood discharge and will tap the reservoir earlier than will the level crest at R.L. 155·00 which has been taken into account in the calculations.

The object of increasing the reservoir level above that of the first closure is to effect the raising of the dam gradually, so as to avoid leaving too much masonry

and embankment work at the outlet for construction at the final closure. The increase of level will also ensure the more gradual wetting of the whole embankment and enable the part thus submerged to attain practically final settlement before the upper part of the dam is completed on it.

Appendix 13, p. 372, gives a calculation of the flood which can be disposed of by the combined outlet headwall discharge and the reservoir absorption. Starting with a discharge of 5,706 cusecs (which is equal to a run-off of 1·39 inches per day from the catchment, and is a fair small flood), it will be seen that in eight hours they dispose of a total flood equivalent to a run-off of 3·98 inches. Of this, 2·18 inches are passed off by the headwall, and the balance, 1·80 inches, is absorbed by the reservoir. When high-flood level is reached, the discharge of the headwall will be at the rate of 0·42 inch per hour run-off, and assuming that the reservoir remains at this level, the total run-off during the day will be 10·69 inches. The maximum high-flood level may therefore be safely taken as R.L. 169·00.

For this closure the whole of the dam would be raised with full section at least to R.L. 175·00.

139. The Final Closure.—For this the under-sluice section of the waste-weir, 250 feet long (Plate 7, Fig. 3), would be left open, the masonry foundations and super-structure constructed to sill level R.L. 177·00, and the permanent tail channel excavated so as to form a temporary waste-weir channel. Detailed calculations of the flood of which it can dispose, aided by the discharge of the outlet sluices and by the absorption of the reservoir, are not given, but, judging from Appendices 12 and 13, pp. 370–373, the high-flood

level will not exceed R.L. 184·00, at which level the discharge from the permanent waste-weir and the outlet will be 20,624 cusecs, which is equal to a run-off of 0·21 inch per hour from the catchment. For the sake of safety the high-flood level may be taken as R.L. 187·00, when the total discharge will be 32,733 cusecs, or at the rate of 0·33 inch per hour run-off from the catchment. These discharges are exclusive of the large amount of flood-absorption by the reservoir.

If the rate of progress permits, the whole of the outlet headwall and the gap in the embankment at it (Plate 4, Fig. 2) should be completed in one season. If, however, the completion of these works cannot then be effected, these parts need not be raised at first beyond R.L. 190·00: at this level the temporary waste-weir will be thrown out of action which it is desirable should be done as early as possible.

After all the works are completed, the reservoir should be kept at as low a maximum level for as long a time as possible, to permit of the practically final settlement and self-consolidation of the top of the embankment in the dry.

140. Quantities of Work to be done in each Closure.—The following table gives the estimated quantities of each class of work involved in each closure:—

1	2	3	4	5	6	7
Consecutive No	Name of Closure	Crest Reduced Level	Earthwork Cft.	Drystone Cft	Masonry and Concrete. Cft.	Total Work Cft.
1	Preliminary Closure, Gorge embankment gap	124·00 166·00	2,673,000	1,772,000	491,000	4,936,000
2	First Closure, ditto, minimum section	166·00	2,494,900	740,000	130,000	3,364,000
3	First Closure, ditto, complete section	166·00	2,818,000	1,727,000	104,000	4,649,000
4	Second Closure	175·00	4,001,000	49,000	59,000	4,109,000
5	Final Closure, full height	208·00	565,000	9,000	290,000	864,000
6	Total Closure	208·00	12,551,000	4,297,000	1,074,000	17,922,000

In these figures flank embankment work is not included, as that can be completed previously to the actual closures concerned.

Owing to the limited extent of the working area and to the necessity for careful and slow construction, it is not safe to reckon upon a greater annual progress than 5,000,000 cubic feet for each of the closures. For the closure in one operation of another design for an entirely earthen dam of about the same size as that for the Máládevi project, it was estimated that 13,642,000 cubic feet of earthwork would have to be completed in one season, and, after reducing the gap beforehand by side embankments, that 11,036,000 cubic feet would thus have to be completed. It is very doubtful if even the smaller of these two quantities would have been feasible in the comparatively short working season of about 180 days, and the raising during that time of so high a mass of earthwork would not have been conducive to its final stability.

Each of the above described closures will involve the construction of temporary work and its future replacement by permanent work, but the cost of this will not be great.

It is essential for all closures to make out a careful programme of the work to be done each month, and to keep the actual construction as much as possible in advance of it so as to provide for the delays which may occur later on (para. 126 (*d*), p. 175).

XII. Earthwork Slips and their Repair.

141. The Necessity for the Sound Construction of the Dam.—If an ordinary high earthen dam is not properly constructed, it may be liable to sudden and

unforeseen slips, which may lead to its failure, and thus may cause consequent damage to life and property, and may entail loss of revenue and income and expensive measures of repair. It is far better to obviate all chance of failure at some extra capital outlay during the construction of the work than to run any chance of risk for the sake of a comparatively small original economy. Precautionary measures to be adopted in the design and during construction have been described above, and, where these have been taken, the fear of failure will practically disappear.

A slip is the only form of damage which will be considered here. The overtopping of a dam is due to insufficient waste-weir provision, and the failure of an outlet is due to mistakes which are pointed out in paragraph 201, p. 286. A slip may be caused in the following ways, by—faulty foundations; unequal settlement, producing internal stress and subsequent motion; defects in the construction or design of the dam; changes in the chemical and physical constitution of the material of the dam or of the subsoil; and by infiltration of water.

142. Faulty Foundations.—Foundations may be faulty from three causes: they may be unduly compressible or porous, or they may be badly seated. In regard to the first two of these it may be stated that most dry soils can withstand the weight of a dam. Argillaceous, or partially impervious ones, are, however, likely to give when saturated. A deep clay seat is therefore undesirable for a dam; where it exists, it must be rendered as compact and dry as possible on the downstream side, by a series of deep, dry rubble drains parallel to the centre line and with cross outfalls as frequently as can be arranged. On both sides the

slopes of the dam should be flattened so as to secure a wider base, and, if necessary, the natural soil below it should be replaced by more reliable material. Berms, also, may be added at the bases of the slopes to prevent the rise of the subsoil (paras. 74 (*b*), p. 108; 76, p. 111; and 114, p. 162).

If the dam is constructed on a stratum which is tilted and rests on a lower one with which it is not firmly united, the extra weight thus put upon the upper stratum may cause it to slide and carry the embankment with it. Careful geological investigation is necessary to avoid this source of danger, as it cannot usually be remedied, except at great expense, by the construction of foundation bonding works.

143. Unequal Settlement.—Wherever the longitudinal section of the dam varies greatly in height, and wherever the construction has not been uniform and slow, or junctions have been formed, there is a risk of the occurrence of unequal settlement, which will cause internal stresses and subsequent motion. The effect of these causes will be intensified by the action of the water, which may thereby percolate from the reservoir, thus saturating the earthwork and forming a slipping plane. The place where this process will most frequently occur, is that where the dam was closed, should this have been done in the usual way at the river crossing (para. 132, p. 183). Slips have occurred in a very few Bombay dams at this place, and years after their construction, showing that some slow disintegrating action, such as that described has been at work (see also para. 118, p. 165, and Appx. 17^{B}, p. 383).

144. Defects in Construction and Design.—The use of too permeable materials will cause a breach rather than

a slip ; but with proper care in construction, ordinary permeable materials become too compact to allow of excessive and dangerous percolation. Friable materials too loose to bind and totally wanting in cohesion, may form a slipping plane and lead to failure. Pure clays are dangerous in that their cohesive and frictional resistances become very largely reduced when they are charged with water, and a too liberal use of water during construction is, therefore, to be strongly deprecated. Through a defective design the earthwork may be unable to support its own weight, and will slip so as to assume more suitable slopes. The profile which is only just sufficient for a dry embankment will prove too slight when it is subjected to water infiltration.

145. Changes in Chemical Constitution.[1]—Some materials are liable to changes in chemical constitution, leading either to deterioration or to disintegration. Occasionally, clay is found to be hard and tenacious originally and to maintain this character as long as it is under water, but, after having been allowed to dry and having been again placed under water, to turn into an impalpable mud, showing that some change has taken place in it ; this is probably owing to some salt, such as carbonate of soda, contained in it. Soils which show an efflorescence on their surface should therefore not be used in the construction of a dam.

146. Infiltration of Water.—This is the principal cause of slips, and it operates in aiding all the others to produce failure.

Undisturbed and compact soils allow water to permeate only slowly and regularly through them,

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[1] "Minutes of Proceedings of the Inst. C.E.," Vol. cxxxii., p. 211. *Engineering News*, Vol xlix , No. 2, dated January 8th, 1903, p. 38.

but they may become fissured by desiccation, or, when artificially formed, by subsidence; the fissures thus produced will be dangerous in that they will allow percolation water to gain powerful hydrostatic pressure in them. When that occurs, such water will endeavour to find an outlet, and if successful in this, may be able to detach from the main part of the embankment the portion of the earthwork thus separated and cause it to slip. Failing to find a defined outlet, the water will tend to saturate the base of the dam and render it too soft to support the mass above it, and again a slip may take place.[1]

It is therefore of the utmost importance thoroughly to consolidate an embankment (para. 119, p. 165), for water will easily find its way between unconsolidated particles of earthwork and may thus produce a defined line of internal flow. It is also important to prevent the formation of a plane of separation such as might result from diagonal drains down the slope of the dam (para. 76, p. 112), from a system of uneven drainage of its base (end of para. 112, p. 160), or from a dry layer (para. 118, p. 165). The rate of settlement of the earthwork should be retarded as much as possible to obviate the formation of slipping planes (para. 74 (*a*), p. 107, and 123, p. 172), and the dam on its down-stream side should be made of self-draining material to avoid the accumulation of percolation water in its interior (para. 69A, p. 99, and 110, p. 155). The surface of the slopes of the embankment should be made as compact as practicable to lessen percolation into the heart of the earthwork, and that of the down-stream slope should during construction be rendered as impervious as possible (end of para. 117, p. 164),

[1] "Minutes of Proceedings of the Inst. C.E.," Vol. lv., pp. 338–41.

should be properly maintained to shed the rainfall evenly, and should be turfed to protect the earthwork from disintegration by the action of the weather (para. 124, p. 173).

Whenever in filtration from the reservoir is excessive and consequent repairs have to be effected, the water level of the reservoir should immediately be lowered to the extent found necessary, in order to prevent a slip or a breach, no matter what temporary loss to irrigation may thus be occasioned.

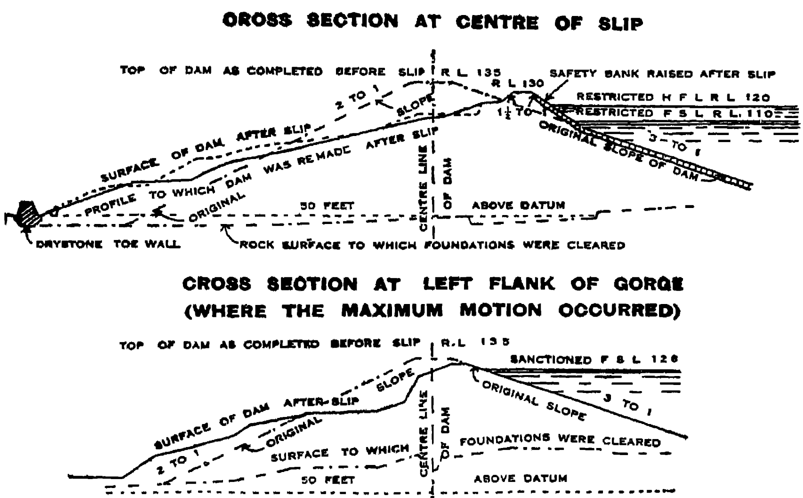
147. Profile of Slips.—Although Indian dams are carefully constructed, still slips of them occasionally occur. Frequently the initial cause is obscured by the fact that several causes are at work together. These slips, when extensive, are of the same general form—S-shaped in vertical section, with the concave portion at the top and the convex at the bottom (when looked at from downstream), and in plan, convex to the original toe of the dam (Fig. 17). This profile [1] is not likely to have resulted from mere disintegration of the earthwork, as such disintegration would probably occur at the surface of the slope of the dam rather than in the interior of the work. It would be fully accounted for by hydrostatic pressure acting in fissures as described in the last paragraph. The layers at the base of a slip are generally tilted at an angle of about 20° to the horizon, so that their inward thrust preserves equilibrium with the outward one of the dam (para. 121, p. 170).

The slip of the Wághád Dam in the Nasik district of the Bombay Presidency, was due to unusual causes. It is illustrated in Fig. 17, which also shows the temporary restoration works carried out before the

[1] "Minutes of Proceedings of the Inst. C.E.," Vol. lv., p. 339.

dam was made up to its final section. The early history of this dam is given in greater detail in Appendix 17B, p. 383, as it furnishes the reasons for many of the recommendations made in this book. In 1883, the year previous to its occurrence, the outlet tunnel on the left bank was built very late and the dam, (which was of pure black "cotton-soil" with only thin muram casings), had to be raised very hurriedly over it. Early in the monsoon of that year the gorge embank-

FIG. 17 WAGHAD DAM SLIP



ment, which had not been raised to the designed height (maximum, 95 feet), was topped by the reservoir and a large portion of its right flank was carried away. In 1884 the dam was restored across the gap and up to 8 feet below the designed top, but the high junction of the old with the rapidly raised new earthwork was a plane of great weakness, and the earthwork on both sides of the junction was in unstable equilibrium. On the morning of April 28th, 1884, a hair crack was seen

travelling up the slope of the dam above the outlet culvert; by the evening it had returned from the top to the base so as to form a loop, and during the night occurred the slip of 200 feet—practically the whole length—of the down-stream slope of the gorge embankment. (Compare the Necaxa slip, para. 57, p. 80.)

148. Slips in Pure Black "Cotton-Soil."—Where a slip has occurred in pure black "cotton-soil," the surface of the dam left standing will present a smooth, unctuous appearance, striated by the small particles of grit in the subsided earthwork, and parallel planes of similar surface will be formed for some feet on both sides of it. It is doubtful if these will ever disappear of themselves; the result will be that the fallen mass will rest on a series of steeply-tilted, smooth, lubricated planes, and a small additional weight, produced either by adding more material to it, or by the soakage of rain-water, will tend to cause further motion. The whole of the slipped earth will also have lost all the consolidation artificially given to it during construction. It will be traversed by minor slipping planes and by fissures, which will admit rain-water and cause it to have a greatly reduced frictional and cohesive resistance to motion. No dependence can therefore be placed upon it, and the only sound system of repair will consist in entirely removing it and in replacing it by trustworthy material.

149. Slips in Gritty Soil.—Where a slip occurs in earthwork having proper proportions of clay and grit, the latter will enable the whole to reunite gradually, but the junction always will be a plane of weakness. This property of reunion of gritty soils is, however, a valuable one. The slipped portion will also be fissured like one of pure clay, but the gritty particles will admit

of slow, self-drainage, and give bond, so that, after a couple of years, the earthwork will attain a fair amount of self-consolidation. The fallen mass when properly drained may, therefore, be utilised in reconstruction, but care should be taken to weight it very gradually, so as to let each season's work attain a practically final consolidation before that of the next one is raised upon it, and to maintain instrumental observations to detect the smallest motion. When this occurs, further additional weight should not be put on until the toe is properly buttressed.

150. Repair of Slips—Preliminary Operations.—The first operation in all repairs of dams formed of gritty soil is to drain the slipped earthwork by drains at right angles to the axis of the dam. Of these one should be at the centre of the disturbance, if this is extensive, and two should be along the junctions of the slip with the solid embankment; others should be inserted at convenient, intermediate distances. They should be taken out in timbered trenches, extending from the toe of the slip to a little beyond its innermost slipping plane, and should extend vertically throughout the mass. As soon as the excavation of each is completed, a longitudinal base drain with a vent 4 inches broad and 6 inches deep should be laid, and the trench should be filled with drystone, having gravel casings at the sides and a 2-foot cover of fine stuff and earth at the top; otherwise it will rapidly choke. Not only do these drains serve their initial purpose of passing out soakage water harmlessly, thus allowing the fallen mass to consolidate itself, but they also divide it into independent sections. It is unlikely that all these sections will tend to slip at once, and thus each at the time of the initial tendency to motion will be supported by

the resistance of a length of the toe works considerably longer than itself. Where practicable, and when the fallen mass has been drained, the whole line of the slip should be followed up and cut out so as to get rid of the slip planes. This may be done in a difficult case by means of a timbered trench. The refilling should consist of gritty self-draining clay, with a good base drain communicating with the cross drains above mentioned.

151. Repair of Slips—Final Operations.—The last step to be taken is to construct a strong, well-drained drystone wall, parallel to the axis of the dam, and with good batters against the slip (Fig. 4, para. 77, p. 113), so as to give the new earthwork the needful increased stability, for it will not have, bulk for bulk, the resistance of the original construction. If the fallen earthwork is to be entirely removed, this wall may be placed at about the centre of the width of the slip; if that is to be allowed to stay, it should be at its toe. To add to the stability of the repair, a strong and well-drained earthen berm should be placed just beyond the original toe of the dam, and its own toe should be secured by a second drystone wall. All these drains and walls should, if possible, be founded on rock, but, where this is not to be found at a reasonable depth, they should be carried well into the natural subsoil and beyond the limit of disturbance caused by the slip. Earthwork usually fails at one point, and, as soon as it commences to move, drags the adjacent parts with it. The drystone walls, instead of transmitting the pressure directly behind them, distribute it over a certain increased area, and this tends to prevent the initial motion. They thus act like the timbering of a trench, which, although incapable of

resisting the full lateral pressure of earth-work, provides sufficient support to prevent the first tendency to slipping.

All open excavation should be carefully taken out in sections, with sufficient widths of undisturbed material between them to act as buttresses, and should be filled as quickly as possible. This, of course, results in the filling not being so well compacted as if the whole length were dealt with at once, but in time this defect will be remedied. It also leads to the formation of numerous junctions, but, as the sections will be constructed within a short time of each other, the earthwork on each side of them will eventually unite in a practically solid way. The slopes of the finished surface should be arranged so that the rain falling on them will be shed uniformly and will not be concentrated irregularly so as to gutter the dam.

It will thus be seen that in dealing with a slip every means should be taken thoroughly to drain the earthwork and to cut out slipping planes and replace them by fresh material having the maximum amount of resistance to movement.

XIII. Pitching.

152. The Necessity for Pitching.—In a large reservoir the action of the wind on the water surface will cause the formation of waves, which, if allowed to break on the unprotected surface of the dam, will very soon wash away the earthwork. To protect that surface it has to be covered with pitching of a perfectly durable character. This is laid on a 6-inch layer of hard, sound muram, or quarry spauls, so as to stop the entry of vermin and the growth of plants, and so that any water, which finds its way through the interstices of

the pitching, may be prevented from washing out the clayey material of the dam, for this would cause the pitching to subside and no longer to present an unbroken surface. As subsidence of the dam will have a similar disturbing effect on the continuity of the pitching, it is desirable, whenever feasible, not to lay the stones until the earthwork has practically attained its final consolidation, for which reason the pitching should be constructed as late as possible.

In Appendix 21, p. 445, are given tables for the estimation of the pitching of a dam embankment.

153. The Laying of Pitching.—The most economical form of efficient pitching consists in stones laid with their broadest ends on the dam, and roughly hammer-dressed to meet for a depth of 3 or 4 inches all round their bases so as to form a complete covering. The stones should break joint in every direction, and long, continuous joints should be avoided, as these may tend to cause the pitching to slip in sections. Owing to the irregularity of the stones the work should be carefully carried out and inspected. In some of the earlier Bombay dams the stones were laid with their broadest ends upwards, and their tops were made to fit each other so as to produce a smooth surface. This form presents a more regular appearance than does the more recent one, but, as in it each stone rests on a small base and may not be properly bedded, such pitching is liable to be displaced by the impact of waves.

Where roughly-squared stones are cheaply procurable, they can be laid in regular courses breaking joint with each other, and this forms the best kind of drystone pitching.

After a section of the pitching has been laid, it should

be tested by a heavy hammer to see that each stone is solidly bedded and firmly bonded with the rest. Each interstice which is left should then be filled by the largest sized chip practicable, each chip being driven well home by a hammer, and, thereafter, the pitching should again be tested.

The following are the defects to be avoided in pitching :—unsound material; larger base of stone laid uppermost; long, unbroken joints; stones set with the longest dimensions of their bases not roughly parallel to the centre-line of the dam; sharp abutting edges involving large interstices; stones projecting irregularly; loosely-placed stones; partial bedding of base of stones; bad fitting of bases of stones; loose packing and incomplete filling of interstices.

154. The Extent and Thickness of Pitching.—The pitching should extend from 2 feet below outlet sill level to 2 feet above the level of the anticipated maximum wave-wash at high-flood level. The latter depends upon the height of the waves, the nature of the surface of the slope of the dam, the alignment of the dam with respect to the direction of the wind and the force of the wind at the dam. Stevenson's formula [1] for the height of waves H, in feet, with a "fetch" F, (in nautical miles of 6,083 feet), or distance for which the wind acts on the water, is :—

$$H = 1{\cdot}5\sqrt{F} + 2{\cdot}5 - \sqrt[4]{F}$$

This formula gives the total height of the wave from its trough to its crest. Assuming that half of this is above and half below still-water level, the top of the pitching should be raised $\frac{H}{2}$ on this account. In

[1] "The Design and Construction of Harbours," Thos. Stevenson, 3rd edn. 1886, p. 29.

addition must be made allowances for the other conditions mentioned above. In regard to the nature of the surface, rough pitching offers the greatest resistance to the travel of wave-splash and smooth concrete the least ; the amount of travel will be in direct proportion to the local force of the wind on the crest of the waves. In respect to alignment the waves will reach the dam at the height due to the fetch but the effect of the wind on their crests will vary with the sine of the angle the dam makes with its direction which alters the length of travel of the wave up the pitching. Thus, when the dam is parallel to the wind, sine $0° = 0$, and the vertical travel will be *nil*. When it is at right angles, sine $90° = 1$, and that travel will be at its maximum. Unless the angle which the wind makes with the dam is constant when the reservoir is full, no reduction of the height of the pitching on this account can, however, safely be made, but the maximum amount of travel possible must be provided for.

The thickness in feet of the pitching at different levels may be taken as somewhat more than one-third of the height of the waves at those levels. Practically it would vary in the case of moderate sized reservoirs from 6 inches at the bottom to 18 inches at the top, and, in the case of the largest reservoirs, from 9 inches to 2 feet. In all cases the thickness must be formed by single stones of the full depth of the pitching, for, if it were made up of two layers, the upper layer, when under the impact of the waves, would tend to be washed off from the lower one owing to the friction between the two being diminished by the lubrication of the water.

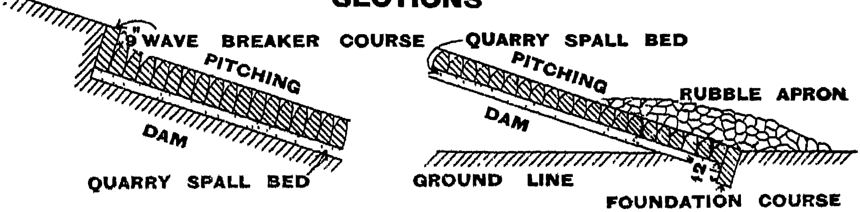
155. Foundation and Top Courses of Pitching.— The foundation course of the pitching should consist

of large stones sunk at least 12 inches into the dam or into the natural ground. Where the toe of the dam is above outlet level and the soil consists of soft material, that should be protected from being undermined by wave-wash by an apron of rubble *débris*, etc., from the various excavations connected with the main work (Fig. 19).

The top course should be formed of hammer-dressed headers with comparatively thin joints so as to prevent the upper part of the dam from guttering through them under the action of rain. This course should be laid in a continuous line parallel to the top of the dam so

FIG. 18 **FIG. 19**

SECTIONS



as to give a finish to the pitching, and should project at least 9 inches above the general surface so as to act as a wave-breaker course (Fig. 18).

158. Concrete Packing of Pitching.—The form of drystone pitching described above is the cheapest one which is reliable, but it has not a very sightly appearance. It is liable to have interstices, in which the percussive action of the waves may exert great force which will tend to blow out the stones, and through them the water will readily gain access to the surface of the dam and may erode it. These interstices may permit of the growth of plants near and above full-

supply level, which may disturb the pitching, and they may allow of the entry into the dam of rats and crabs, which may thus be able to burrow and do a great deal of harm not noticeable from the surface. The projecting tops of the stones permit the waves to exercise a considerable amount of leverage in disturbing the pitching. The proper inspection of such pitching is at all times difficult.

These defects would be remedied were the pitching packed to a fairly uniform surface with fine concrete, or very coarse mortar, worked into all the interstices so as to unite the whole into a solid covering. To allow for settlement, this packing should be completed in detached sections, say 10 feet long and 5 feet wide, breaking joint with each other. The packing should be carried out a year or two after the reservoir has first filled, (by which time the earthwork of the dam will have practically attained its final settlement), and while the water level is falling, so as to permit, at small cost, of the mortar being kept wet in order to set properly. This form of pitching would, of course, be dearer than ordinary dry pitching, but it would be more durable and sightly, and would be more easily inspected and maintained in good order.

157. Concrete and Brick Pitching.—On some works [1] in England pitching formed of concrete blocks has been tried, but in India this would probably be a much dearer form than stone, as the latter will usually be available at a cheap rate at dam sites. Moreover, concrete blocks will not be so durable as stone, and will be liable to wear at the joints. It would seem better if concrete has to be adopted, to form it *in situ* in large slabs, say

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[1] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii., p. 226.

10 feet long and 5 feet wide, and separated from each other by free joints in order that it may accommodate itself to the settlement of the dam without at the same time exposing numerous joints to the action of the waves.

Brick pitching has been used on some river embankments in Sind and might be tried on small reservoirs where stone is not available, being laid stretcher-wise for small and header-wise for moderate sized reservoirs. Bricks are scarcely hard enough to resist the impact of large waves, and cannot be made of sufficient depth to withstand it. Like stones they should be laid in a single layer.

158. Grass or Reed Revetments.—In some Madras [1] tanks a grass, or reed, revetment is used. The reed employed is one that grows in marshy land, and has numerous joints, at each of which is an eye, from which a shoot will spring if it is buried in the ground. A layer of reed is first placed horizontally on the embankment with the root ends inwards; along the centre of this a fascine made of the reed is placed parallel to the axis of the embankment, thus forming a step; the projecting part of the layer is folded over it and is covered with soil, and subsequently other steps are similarly formed. The whole is then watered until the eyes at the joints of the reeds begin to shoot. Eventually a forest of reeds, 10 or 12 feet high, springs up and forms an effective barrier to the action of waves.

This is, of course, the cheapest form of protection possible, but it is open to the great objection that the surface of the embankment is entirely hid from inspection, and thus rats and crabs may burrow in it

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[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 92.

unperceived and unsuspected. It is an essential for a covering of an embankment of any height that it should be fully open to view, so that any damage to it, or to the underlying earthwork, may at once be detected and repaired.

In Sind, river embankments are sometimes protected by allowing tamarisk to grow a little distance in front of them, and, in the case of high embankments, up the base of the slope, care being taken to trim off all bottom branches so that the earthen surface may fully be exposed to view.

Neither of these forms of protection by natural growth is, however, suitable to high reservoir embankments.

CHAPTER III.

THE WASTE-WEIR.

I. The Weir Proper.

159. Different Forms of Waste-Weir.—Waste-weirs are usually of three forms :—

(*a*) Drowned channels ;
(*b*) Drowned weirs ;
(*c*) Clear overfall weirs.

These three classes of weirs are simple and perfectly automatic in their action, and the selection of one of them depends upon the nature of the ground and upon its levels. For all classes the bed of the upstream approach channel if excavated, is level, and practically at the same level (para. 164, p. 217) as the actual crest of the weir, while the downstream tail channel is made with a slope falling from the weir crest, so as to provide for the flow of the flood with a small tail depth ; the inclination of this slope is usually made 1 in 100 when the ground is sufficiently hard to withstand the velocity thereby produced (para. 179, p. 247).

The following table shows the discharging power per foot run of weir of classes (*a*) and (*c*) :—

Discharge of Waste-Weirs per Lineal Foot in Cubic Feet per Second.

Total depth of Flood in Feet	1	2	3	4	5	6	7	8	9	10
Drowned Channel with fall of 1 in 100 (approximate)	2·38	7·29	13·70	22·22	32·55	43·92	57·04	71·87	87·92	106·05
Clear Overfall Weir . .	3·29	9·39	17·47	27·16	38·25	50·52	63·87	78·22	93·55	109·78

The drowned channel discharges are taken from Appendix 10, p. 366, and the clear overfall discharges, from Appendix 11, p. 368.

160. Drowned Channels and Weirs.—A drowned channel weir (Fig. 25, p. 236) is one having the crest of the weir at its bed level. Where the natural surface is of rock, or hard material, at, or above, fully-supply level, the channel can be left as a simple excavation. Where, however, there is any likelihood of erosion, the actual crest of the weir should be constructed as a wall founded on a reliable stratum so as to preserve the full supply-level of the reservoir, and so as to distribute the flood discharge evenly over the whole width of the tail channel. Without such a crest wall, one or more deep scour channels might be rapidly formed by the concentration of the tail discharge down them. The channel form with a gentle longitudinal tail slope is best suited to soft soils, so that the severe action of an overfall on them may be avoided.

A drowned weir (Fig. 26, p. 241) is one of small height, having its crest below the surface of the tail channel discharge when the reservoir is at high-flood level.

161. Clear Overfall Weirs.—A clear overfall weir (Fig. 27, p. 242) is one having its crest above the surface of the tail channel discharge when the reservoir is at high-flood level. As the action of the overfall on the foundations is very severe, the weir wall must be founded quite securely. The best foundation is one of solid, unfissured, compact rock, into which the waste-weir wall should be founded for at least 2 feet in depth. Where this material does not exist, the wall must have its foundations carried considerably deeper and at least to 5 feet in depth; soft, erodible soils must be

protected by a water-cushion (Fig. 24, p. 216) which will also lessen the initial horizontal velocity of the water as much as possible.

Where the natural levels and formation of the ground permit, it is advisable to convert drowned weirs into clear overfall ones, so as to gain increased discharging power. It is, of course, not necessary for this purposes to lower the tail channel more than the depth of the high-flood discharge down it; in fact, where the ground is of a soft nature, a less amount of excavation will suffice, as the floods will themselves scour out the channel. The allowance for this scouring action can safely be made by deepening, to the full extent, and full bed-slope, only the head of the tail channel for a length sufficient to prevent the surface of its discharge from topping the weir, the bed of the rest of the channel being continued therefrom with a less inclination (see also para. 177, p. 244).

In a few old native tanks where the foundations are bad and the ground is considerably below the full supply-level, the waste-weir has been formed as a rapid; but this has not been done in the case of modern works of any size, as rapids may entail much expenditure on maintenance. A natural rapid of hard material does not, however, involve as much expenditure on maintenance as does an artificial one, for excessive erosion of it can be prevented by cross curtain walls (para. 178, p. 245).

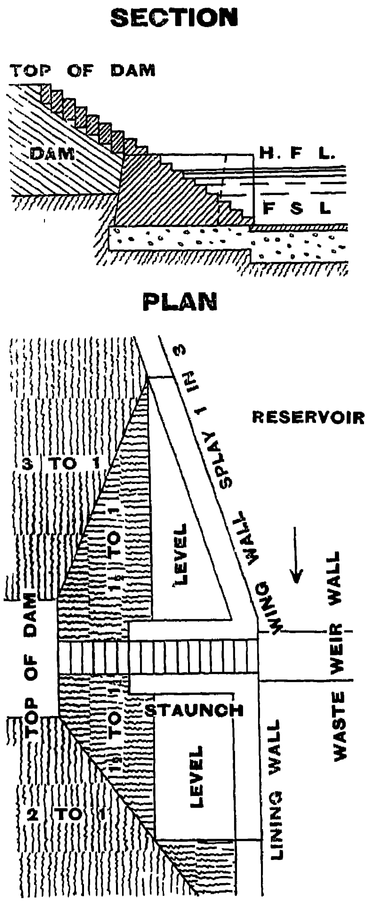
162. Positions of Weirs—Flank and Saddle Weirs.—In respect to their positions waste-weirs may be classed as:—

(*a*) Flank weirs, at the immediate flank and in continuation of the dam embankment;

(*b*) Saddle weirs, separated from the dam by high ground.

Flank weirs are naturally the less safe form of the two, as the discharge from them is liable to out-flank and injure the dam. To prevent this a wing wall is required on the upstream side and a lining wall on the downstream side. The lining wall is necessary to divert the floods from the dam; it should be continued until the tail channel has a clear course away from the embankment, and should be raised at least 2 feet above the surface of the tail channel high-flood discharge. The junction of the waste-weir wall and the dam is best effected, as shown in Fig. 20, by a stepped passage, which permits of easy access to the works and at the same time acts as a staunching fork, preventing leakage from occurring at the junction.

FIG. 20



Saddle weirs do not require such protective works, as they discharge clear away from the dam, but, to prevent them from being out-flanked, they should have masonry flanks on each side, raised at least 2 feet above high-flood level, and flood embankments in continua-

tion, raised at least 5 feet above that level. As explained in paragraph 75, p. 110, these flank embankments can be utilised as "breaching sections" to prevent the over-topping of the main dam embankment during abnormal floods. Similar flank works are required for flank weirs at the side remote from the dam, when the ground there is below high-flood level so as to direct the tail flood down a defined channel.

Saddle weirs are decidedly the better form, and should be adopted wherever practicable in preference to flank weirs, as the waste-weir tail channel should discharge as far away as possible from the dam.

163. Sections of Weir Walls.—There is no advantage, so far as discharging capacity is concerned, in raising the crest of the waste-weir wall above the high-flood level of the tail channel, and such additional raising has the great disadvantage of increasing the action of floods on the foundations of the weir. Waste-weir walls will, therefore, generally be low, and will be high only when the ground where they are constructed is much below the designed full-supply level of the reservoir.

Fig. 21 shows a form which may be adopted for the crest wall of a channel weir with good foundations; Fig. 22, one for a channel weir with bad foundations; Fig. 23, one for a clear overfall weir with good foundations; and Fig. 24 one for a clear overfall weir with bad foundations.

Where there are not to be any weir-crest shutters, the top-width for low weirs should not be less than 2 feet 6 inches; where such shutters are used, the top-width will have to be increased to permit of their being worked. The upstream face of the wall should be vertical, and its downstream one should batter

1 in 4. Preferably the downstream face of a weir with deep foundations should have the batter continued as a masonry facing to protect the concrete base. The crest should always be level in cross-section, as it is not necessary to ramp it down to the reservoir side to help the passage of silt over it, as is desirable in the case of a river weir (but see Appendix 11, note 5, p. 369). The downstream edge of the crest should be corbelled

FIG. 21

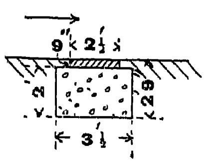


FIG. 22

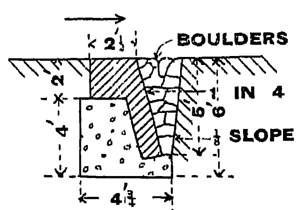


FIG. 23

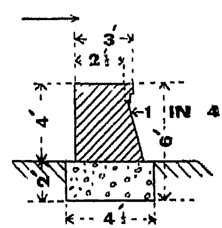
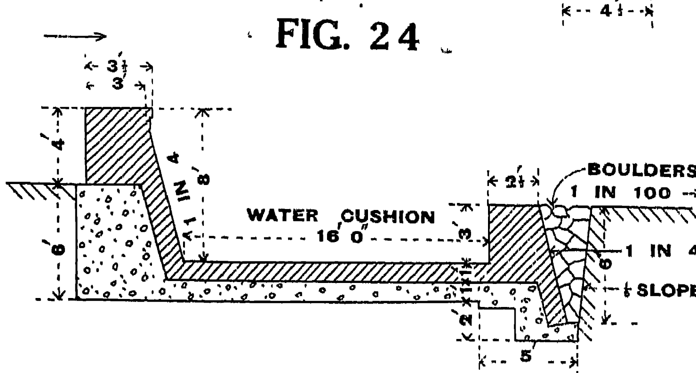


FIG. 24



out so that the floods passing over it may be made to discharge clear of the downstream batter of the weir. The mean width of weirs (built of heavy trapstone masonry) of a less total height than 10 feet may be taken in ordinary cases as equal to two-thirds their height: for such weirs of greater height, and for those discharging a great depth of flood, their sections should be determined by means of stability diagrams.

In a recent Italian design the waste-weir wall is formed as a siphon which comes into action a little below high-flood level. It is claimed that the high flood is discharged through the siphon with only a very small rise in the level of the reservoir, so that the height and the cost of the dam can be considerably reduced. It is believed that this design has not been carried out on a large scale.

164. The Approach Channel.—This is usually excavated level with the crest of the weir, but, where the channel is very long and the water in passing over it would thus lose head by the friction of the bed, it is preferable to excavate it from 6 inches to 12 inches lower than crest level.

The approach channel must have a perfectly clear and unobstructed course of the full width to the waste-weir, as any contraction of it would cause the water to head-up upstream, and would thus diminish the discharging capacity of the waste-weir crest. In determining that capacity the effective length of the waste-weir crest to be taken into account is that which is measured normally to the line of flow. It is of no use curving the weir crest, or making it oblique in plan in order to increase its length beyond this amount, as that will not lower the reservoir below the level necessary at the narrowest part of the approach channel to pass the required discharge. Such an increase of length will diminish slightly the flood depth over the crest itself, but will thereby induce a greater velocity of approach to it, and the head thus consumed will practically be equal to the diminution of the flood depth over the weir crest. Were this artifice of increasing the length of the work effective, it could be developed greatly by making the weir

deeply serrated in plan, and this at once shows that it is not really of any use.

To flow with full effect over an oblique weir, the water would have to change its direction so as to cross the weir at right angles, and therefore to make that work properly effective, the approach and tail channels would have to be curved,[1] and thus widened correspondingly; consequently, the oblique weir would practically become one at right angles to the altered channels.

The amount of excavation of the approach channel of a flank weir may, however, be considerably decreased, where necessary, by curving it, as shown in Fig. 28, paragraph 177, p. 244, as this will save the removal of the soil between the outer curve and the line at right angles to the end of the weir there (where the surface of the ground will usually be of the maximum height) without diminishing the normal width of the channel. Care must be taken, however, not to make the curve too abrupt, or else the high-flood water will have an irregular flow over the weir crest, which will thus be less effective in discharging it. For this reason the approach channel curves should be set out from a centre on the centre line of the dam some distance away from the waste-weir.

165. Safety Flood Cuts.—Where the ground at the proper level is sufficiently hard, auxiliary flood cuts can be excavated so as to help the discharge of the main weir in times of abnormal flood. The level to which these should be excavated is one which will let them come into play only during high floods, so that they will not be exposed to the long-continued action of

[1] "Minutes of Proceedings, Inst. C E," Vol. lx., p. 114.

moderate floods; they will thus usually not require expensive protective works to prevent their erosion. Generally speaking, that level may be 2 feet below the calculated high-flood level. Such flood cuts will not have a great discharging power per foot run, and, therefore, to make them of substantial assistance to the main weir they must be of considerable length. They should, of course, be situated where their discharge will not do any injury.

That discharge cannot be utilised for irrigation, partly because it will take place at rare and uncertain intervals, and partly because, when it does occur, it will be the result of excessive rainfall, rendering irrigation unnecessary at the time. From a main waste-weir having prolonged flow an irrigation channel might be led off by a pipe, or cut, somewhat below full-supply level and beyond the reach of the waste-weir discharge, so that it might thus utilise for the irrigation of high lands water which would otherwise run to waste.

166. The Depth of the Maximum Flood on the Waste-Weir.—The depth of the maximum flood permissible on a waste-weir depends upon the nature of its foundations; the sounder they are, the greater may be the depth of flow. A deep discharge, of course, lessens the length of the waste-weir required, and, if adopted, may somewhat diminish the cost of the work. However, a long weir is safer than a short one, as any abnormal increase of the depth of the flood over it will result in greater discharging power than the same increase of depth will do over a short weir.

As stated in paragraph 78, p. 114, the less the flood depth over the waste-weir, the lower will be the height required for the dam, and, consequently, the cheaper

will be the embankment. Comparative estimates of the combined costs of waste-weirs of different lengths and dams of different corresponding heights should, therefore, be made, and, other things being equal, the cheapest and safest combination of the two should be selected.

The depths of the maximum floods generally allowed vary from 4 feet to 6 feet, and the weirs are, therefore, of great length. This margin of safety between full-supply level and high-flood level involves what may be considered, either as a very expensive, but essential addition to the cost of the dam, or as a surrendering of a large amount of storage. In a few cases the temporary raising of the reservoir level by a small amount at the end of the monsoon has been attempted, either by means of low earthen banks, or by removable teak shutters fitted into removable rolled joist uprights. The length of the weirs makes this expensive, and, what is worse, slow in management, but the usual design of the works renders it difficult to arrange for automatic or quick-acting, devices for discharging floods (paras. 184, 188, and 189, pp. 253, 260, 261), and without them such temporary raising of the reservoir surface is attended by an undesirable amount of risk.

167. The Factors producing the Maximum Rate of Run-off.—In paragraph 7, p. 10, the principal factors influencing the amount of the yield from a catchment were described. These factors have a similar influence in determining the amount of the maximum rate of run-off which may be expected. Each catchment has a maximum rate of run-off depending upon its local conditions, but, it may be stated as a general rule, that the smaller the catchment the greater will

be the intensity of its run-off, other conditions being the same. A very small catchment may be subject to intense rainfall falling at the same time over its entire area, and the maximum rate of run-off from all parts of this will probably reach the waste-weir at practically the same time. As a catchment increases in size, the less likely is it to have such intense rainfall occurring simultaneously over its whole extent, while the maximum rate of run-off from the more distant portions will probably reach the waste-weir later than that from the ones nearer to it. Thus the average rate of run-off from a large catchment will generally be less than that from a small one (para. 171A, p. 235).

Heavy rain often falls over very large areas at the same time, and lasts for long periods, so that the flood discharges of the upper tributaries of the reservoir have sufficient time to arrive at the site of the dam simultaneously with those of the lower streams. This widespread and long-continued rain is, however, usually of less hourly intensity than that of sudden local storms, but its effect is to make the rate of the maximum run-off from large catchments vary less in proportion to their size than that from small catchments.

168. Formulæ for the Rate of Maximum Run-off.—There are several formulæ for the rate of maximum run-off; the best known and most frequently used of these are given below.

(*a*) *Dickens' Formula.*—This formula is :—

$$D = C\,M^{\frac{3}{4}},$$

where D is the flood discharge in cubic feet per second;
M, the catchment area in square miles; and
C, a coefficient varying from 150 to 1,000 or more, and generally taken as 825.

This formula does not take directly into account the intensity of the rainfall. It allows for the greater rate of discharge from small than from large catchments, and agrees fairly well in this respect with the table of waste-weir runs-off in Appendix 8, p. 363, as will be seen by the following comparison :—

1	2	3	4	5	6
	By Dickens' Formula.				
M = No. of square miles in catchment.	$M^{\frac{3}{4}}$	Relative Run-off per square mile	Comparative Run-off per square mile	Allowance for Run-off in Appendix 8	Remarks
			In inches per hour	In inches per hour.	
1	1·00	1·00	3·00	3·00	Col. 3 = $\frac{\text{Col. 2}}{\text{Col. 1}}$.
5	3·35	0·67	2·01	2·36	
10	5·62	0·56	1·68	1·95	Col. 4 = Col. 3 × 3, in order to make its initial run-off allowance the same as that of Col. 5.
20	9·45	0·47	1·41	1·51	
50	18·79	0·38	1·14	1·10	
100	31·62	0·32	0·96	0·89	

(*b*) *Ryves' Formula.*—This formula, which is much used in Madras, is :—

$$D = C M^{\frac{2}{3}},$$

where D is the flood discharge in cubic feet per second ;
M, the catchment area in square miles, and
C, a coefficient which is taken thus :—
within 15 miles of the coast . . = 450
from 15 to 100 miles from the coast . = 563
for limited areas near the hills . = 675

This formula also does not take directly into account the intensity of the rainfall. It also allows for the greater rate of discharge from small than from large catchments, but does not agree so well in this respect as

does Dickens' with the table of waste-weir runs-off in Appendix 8, as will be seen by the following comparison:

1	2	3	4	5	6
	By Ryves' Formula				
M = No of square miles in catchment	$M^{\frac{2}{3}}$	Relative Run-off per square mile	Comparative Run-off per square mile	Allowance for Run-off in Appendix 8	Remarks
			In inches per hour	In inches per hour	
1 .	1·00	1·00	3·00	3·00	Col 3 = $\frac{\text{Col. 2}}{\text{Col. 1}}$
5 .	2·92	0·58	1·74	2·36	
10 .	4·64	0·46	1·38	1·95	Col. 4 = Col. 3 × 3, in order to make its initial run-off allowance the same as that of Col. 5.
20 .	7·37	0·37	1·11	1·51	
50 .	13·55	0·27	0·81	1·10	
100 .	21·53	0·22	0·66	0·89	

(*c*) *Craig's Formula.*[1]—This formula takes into account the varying width of the catchments, their slopes, and the amount of rainfall (but not its hourly intensity, which is a very important factor), and is therefore theoretically much more exact than the other two. It is :—

$$D = 440\,B\,(C\,v\,i)\text{ hyp. log. }\frac{8\,L^2}{B},$$

where D is the flood discharge in cubic feet per second;
B, the mean width of the area in miles ;
C, the coefficient of discharge, varying with the nature of the basin ;
v, the velocity of the drainage in feet per second ;
i, the rainfall in inches ;
L, the length of the catchment in miles measured from the point of discharge to the centre of the watershed base.

[1] "Minutes of Proceedings, Inst C.E.," Vol. lxxx., p. 201.

For irregular catchments, the perimeter is made regular by equalising straight lines, and the whole area is divided into triangles with their apices at the point of discharge. The total discharge is the sum of the discharges of the component triangles.

168B. The Limited Utility of Run-off Formulæ.—It will be seen that each of these formulæ requires the application of a coefficient, C: this should be determined in each case by experiment or from general practical experience of the behaviour of catchments similar to the one being investigated; it will not suffice to treat the coefficient as a standard multiplier universally applicable without reference to the conditions on which it should depend. It is said no two leaves in a forest are precisely similar to each other; much less likely will catchments, which vary in so many particulars, be exactly the same in regard to their rate of high-flood discharge.

Next, the first two formulæ treat the catchment as a uniform whole: they do not allow for its shape nor for its position with respect to the direction of storms (para. 7, p. 10), the only variation dealt with is that of the area of the drainage basin. They do not take into account the effect of changes of surface slopes nor of the different degrees of porosity of the surface. It would seem from them that during a prolonged and heavy storm it is considered all surfaces, being fully saturated, will act alike in respect to *run-off*: this is not the case, for even then a steep rocky slope will produce a discharge much greater in rate than that from a flat absorbent plain. The *yields* from two such catchments may then be much the same in total amount, but will be given in times inversely proportionate to their rates of run-off: the principal

difference will be due to loss by evaporation and absorption.

Further, these formulæ do not directly include the effect of the intensity of the rain—a highly variable and inconstant factor—and apparently assume all catchments will be visited by storms of equal violence and duration.

It may be said that all these variations of condition are provided for by the coefficient: that would be true only were the coefficient varied for each catchment, or in other words, if each catchment were considered in respect to its individual peculiarities which is precisely what is here recommended should be done.

Craig's formula is superior to the other two as its coefficient deals with the variation of fewer factors, other changes of condition being separately allowed for in it. Another great improvement is that it divides the catchment into constituent areas, determines the discharge of each, and sums these up to obtain the amount of the total flood. A considerable advance in this direction would be made if each constituent area were analysed and dealt with in respect to its individual characteristics of surface, position, and rainfall affecting its high flood discharging capacity (see para. 11B, p. 16).

The factors modifying the rate of high-flood discharge are so numerous and variable, that it is doubtful if any formula can be devised to include all and allow properly for them. Nature, however, does this and gives the correct solution of the problem—the actual resulting flood. The engineer should therefore observe, calculate, and tabulate the information she gives if he desires to obtain reliable statistics of observed high floods which will enable him to design his work with

confidence. It is only in the absence of flood observations that he should resort to a formula for high-flood discharge, and even then should use it with discrimination and allow liberally for extreme conditions. The utility of such a formula is that it enables a general idea to be gained of the probable amount of the highest flood with which a work should be able to deal safely but not at undue cost.

A formula may be most correctly adopted when it has been worked out for a neighbouring catchment, with physical conditions similar to those of a scheme under investigation, from discharges observed for a long series of years. That formula can still be utilised, if the run-off factors of the two catchments are not precisely the same, by obtaining, by means of concurrent flood observations of each catchment, a coefficient which can be applied to the observed discharges of the first to give the probable ones of the second.

To sum up: high-flood discharge depends upon two main factors—the character of the rainfall and the nature of the ground on which it descends. In respect to rainfall the meteorologist admits that his is at present the most inexact of sciences, while in regard to the ground its diversity is patent in hilly regions which are those best suited for reservoirs. The combination of these two variable main factors must produce still greater variation of result. The essence of science is measurement, and the scientist has to discover what to measure and how to measure it: to determine floods he must rely chiefly on observations in the field and not on mathematics in the study.

169B. Flood Observations—Cyclones.—The waste-

weir has to discharge safely the greatest flood which may occur, although such a flood may not happen except at intervals of many years. Hence it is necessary to determine, as accurately as possible, the greatest discharge which has taken place by the observation of actual floods. Such observations should therefore be started as soon as the investigation of a scheme is commenced: thereafter, they should be continued for many years, even after the work itself has been constructed, so as to compile statistics which can be utilised for later projects. It will seldom be possible to obtain a sufficiently long record for a new work, and it should thus be recognised that an extreme flood (for discharging which provision must be made) may exceed the ones gauged. If the record extends for ten years and includes an abnormally high flood, it will therefore be advisable to add at least 10 per cent. to that high flood to arrive at the discharge with which the waste-weir should be able to deal. If the record is shorter, the allowance should be greater, say, by 3 per cent. per year of shortage, up to a total of 25 per cent. excess.

If an extreme high flood has not been gauged, its discharge can be calculated from its flood sections if these are at once taken. If, however, they have not been, they should otherwise be determined, thus:—the height to which such a flood rose will generally be impressed upon the minds of the villagers in the neighbourhood, and usually they will be able to point out its high-flood levels. These points should be tested by levelling both banks, and after verification and adjustment, may be accepted as indicating the true flood sections fairly. To allow for inaccuracy of observation, the discharge thus calculated should

be increased by 10 to 25 per cent. to arrive at the amount of waste-weir provision necessary.

When a reservoir has been designed with flood-absorptive capacity, it is not necessary that the waste-weir should be able to pass the calculated high flood as it arrives, for the storage capacity of the reservoir will act as a moderator of the intensity of the discharge as is explained in paragraphs 182–184, pp. 251–254.

Where drainage areas are liable to be visited by cyclones, it is not practicable to determine exactly what will be the discharge resulting from such storms, as their intensity is so abnormal. In such cases it is best to provide liberally for the ordinary maximum floods, and, in addition, to allow for these abnormal ones by designing breaching sections (para. 75, p. 109), and safety flood cuts (para. 165, p. 218). This will obviate the extra expense of the provision for permanent works sufficient to dispose of cyclonic floods, which may never occur, and will entail only the liability for the much smaller cost of the repair of these temporary works should those floods take place.

In paragraph 28, p. 47, it has been stated that in twenty Bombay tanks the average cost of the waste-weir is only 9·45 per cent. of that of the whole reservoir. As a proper provision for this essential to safety is comparatively so cheap, and as the results of the bursting of a dam may be so disastrous, it is always more prudent to err on the side of safety and to make ample allowance for floods than to run any risk of failure by having insufficient waste-weir discharging capacity.

170. Tanks in Series.[1]—In this connection it may

[1] See "Minutes of Proceedings, Inst. C.E.," Vols. cxxxiv., p. 66, and cciv., p 410; also para. 24B, p. 42 above.

be noted that it is most undesirable to construct a chain of tanks one below the other on the same stream, for the failure of an upper one may cause all the lower ones to breach successively by what, in effect, will be an artificial cyclone. It may be said that modern reservoirs are constructed so as not to breach, which is, of course, true; but accidents to them have occurred, and it certainly does not appear to be sound engineering to run any needless risks in this way. The proper treatment of such a stream is to form on it one reservoir capable of impounding a storage equivalent to that of all the proposed minor tanks. Where, however, from physical conditions a chain of tanks is absolutely unavoidable, care should be taken that the capacities of the works increase in the order in which they stand with relation to each other, *i.e.*, the downstream ones should always have larger storages than the ones upstream of them, so that the former may better be able to absorb the sudden and large inflow which would result from the failure of the latter. Full provision for dealing with abnormal floods should be made, in such cases, by means of breaching sections and safety flood cuts.

In Madras, Ryves' formula (para. 168 (*b*), p. 222) has been adapted for the calculation of the maximum rate of flood discharge from connected catchments, thus:—

$$D = CM^{\frac{2}{3}} - cm^{\frac{2}{3}}$$

where c is taken as $\frac{1}{5}C$; and

m is the area in square miles of the portion of M draining into the upper tanks.

This formula, however, excludes one important factor—the relative sizes of the upper tanks compared

with those of their individual catchments; if this proportion is small, the tanks may have little moderating effect; where it is very large, they may absorb most of the run-off into them. When a tank is filling above its full-supply level a part of the incoming flood is absorbed in raising its level, and the proportionate amount of this depends upon the area of the tank's water-spread, or basin, compared to its catchment area (para. 182, p. 252). To estimate the effect of the reduction of the rate of discharge from an upper catchment by a tank situated in it, the more correct method would appear to be to take into account the area of the upper basin, and not that of the catchment draining into it, and to allow a properly reduced rate of run-off from that basin as the rate of discharge from the waste-weir of the tank. In other words, it would seem better to estimate separately the probable high-flood depth of each minor tank, after allowing for its flood-absorptive capacity, and to calculate the resulting discharge of its waste-weir when determining the amount of waste-weir provision required for the main reservoir.

171. Empirical Allowances for Waste-Weir Runs-off.—Although it is better to depend upon observations for the determination of the maximum discharge for which waste-weir provision has to be made, still in India sufficient experience has been obtained of the behaviour of reservoirs to enable allowances to be prescribed which err sufficiently on the side of safety, and can therefore be adopted without incurring risk.

A table of such allowances for average catchments is given in Appendix 8, p. 363. The hourly rates of run-off from gradual increments of catchment area (col. 2) have been adjusted, so that when plotted in

diagram form (Plate 2), they are on a regular curve; the discharges due to these runs-off for each increment in catchment area are entered in col. 3; the discharges from the total catchment areas are given in col. 4, and from them the average hourly rates of run-off from the total catchments are deduced and noted in col. 5 and plotted on Plate 2. The object of thus determining these average runs-off is to prevent a total discharge from a larger catchment being calculated from the table as less than that from a smaller one, and to obviate the neglect of the effect of the intensity of the rainfall on the smaller constituent areas near the dam. Thus, if col. 2 alone were considered, the amount of run-off from 10 square miles would be calculated as $(10 \times 1{\cdot}40 =)$ 14·00 inches from 1 square mile, and from 11 square miles only to $(11 \times 1{\cdot}16 =)$ 12·76 inches from 1 square mile. Taking col. 5 properly into account, the results are, respectively, equal to:—

$(10 \times 1{\cdot}95 =)$ 19·50 and $(19{\cdot}50 + 1{\cdot}16 =)$ 20·66 inches from 1 square mile.

Special conditions of the catchment area, of course, have to be taken into account; a purely ghát catchment would be given a greater, and a purely plain one a smaller allowance. Allowance for the general intensity of heavy rainfall in the locality and also for the amount of actual floods would have to be made.

171A. High-Flood Run-off from Different Catchments.—On page 233 is a table comparing the rate of the high-flood run-off from, and discharge of, different classes of catchment areas, and the corresponding curves are drawn on Plate 2A. Three values are there given for Bombay (Deccán) catchments, and one each for Ceylon and Transvaal catchments.

The first curve for Bombay catchments is plotted from Appendix 8, p. 363: it shows throughout larger results than those of the other two similar curves: for the smaller areas its increase of discharge is not very great, but for those over 100 square miles in extent is considerable. The additional allowance is made for the sake of providing safety, the necessity for which naturally increases with the size of the catchments and the consequent importance of the works to be constructed on them, as the larger are the reservoirs, the greater is likely to be the damage to life and property which will be caused by their failure. As pointed out in paragraph 167, p. 221, very heavy falls of rain have been known to occur in the Deccán over extensive areas, and it is therefore necessary to provide sufficient waste-weir accommodation accordingly. At Máhilla, near Dhulia in the Khándesh district, the Pánjhrá River on September 15th, 1872, had a flood discharge of 276,000 cusecs, which is equal to a run-off of 0·54 inch per hour, from its catchment of 788 square miles. An extension of this curve would allow for such a discharge, whereas that of the other two Bombay curves would not. Other excessively high runs-off occurred in 1882,[1] and abnormal floods must be arranged for although they may not take place except at intervals of many years. The designed allowances (Appendix 1, p. 347A, cols. 14 and 15) for Ashti (No. 12), Mhaswad (No. 14) and Máini (No. 17) tanks may also be referred to in support of this curve. The discharge estimated for Ekruk (No. 11) is considerably less than that given in the table, but that tank is situated much to the east where there is not great

[1] "Minutes of Proceedings, Inst. C.E.," Vol. cxxxii., Appendix II.

TABLE COMPARING THE HIGH-FLOOD RUN-OFF FROM AND DISCHARGE OF DIFFERENT CLASSES OF CATCHMENT AREAS.

Square Miles	Run-off in Inches per Hour: Bombay (Deccán) Catchments			Run-off in Inches per Hour: Ceylon Catchments	Run-off in Inches per Hour: Transvaal Catchments	Discharge in Cubic Feet per Second: Bombay (Deccán) Catchments.			Discharge in Cubic Feet per Second: Ceylon Catchments	Discharge in Cubic Feet per Second: Transvaal Catchments.	Square Miles.
	Appendix 8	Beale	Whiting			Appendix 8	Beale.	Whiting			
1	3·00	2·50	2·75	1·24	1·50	1,936	1,600	1,775	800	968	1
2	**2·82**	**2·13**	**2 36**	**1·16**	**1·35**	**3,640**	**2,750**	**3,050**	**1,500**	**1,742**	**2**
3	2·65	1·89	2·17	1·08	1·23	5,130	3,650	4,200	2,100	2,387	3
4	**2·49**	**1·76**	**2·03**	**1·01**	**1·14**	**6,421**	**4,550**	**5,250**	**2,600**	**2,935**	**4**
5	2·36	1·66	1·95	0·94	1·05	7,615	5,356	6,292	3,030	3,387	5
6	**2·25**	**1·58**	**1·89**	**0·88**	**0·97**	**8,725**	**6,100**	**7,300**	**3,400**	**3,774**	**6**
7	2·16	1·51	1·83	0·82	0·91	9,770	6,850	8,250	3,700	4,097	7
8	**2·08**	**1·46**	**1·77**	**0·78**	**0·85**	**10,751**	**7,550**	**9,150**	**4,000**	**4,387**	**8**
9	2·01	1·42	1·72	0 74	0·80	11,687	8,250	10,000	4,300	4,645	9
10	**1·95**	**1·39**	**1·67**	**0·71**	**0·76**	**12,590**	**8,970**	**10,777**	**4,600**	**4,871**	**10**
15	1·69	1·26	1·51	0·64	0·59	16,333	12,197	14,617	6,200	5,677	15
20	**1·51**	**1·17**	**1 38**	**0·59**	**0·49**	**19,560**	**15,100**	**17,811**	**7,600**	**6,322**	**20**
25	1·40	1·10	1·30	0·56	0·42	22,528	17,747	20,973	9,000	6,838	25
50	**1·10**	**0·90**	**0·98**	**0·40**	**0·27**	**35,435**	**29,040**	**31,621**	**13,000**	**8,774**	**50**
75	0·97	0·81	0·85	0·32	0·22	47,000	39,204	41,140	15,500	10,387	75
100	**0·89**	**0·74**	**0·75**	**0·28**	**0·19**	**57,500**	**47,754**	**48,400**	**18,000**	**11,838**	**100**
150	0·80	0·61	—	0·23	0·15	77,000	59,048	—	22,000	14,096	150
200	**0·73**	**0·50**	—	**0·20**	**0·12**	**94,000**	**64,533**	—	**25,000**	**16,032**	**200**

likelihood of heavy rain falling all over a large area at the same time, and its catchment is generally not steep. For good sites the cost of ample waste-weir provision should comparatively not be great (para. 28, p. 47), and that cost can be safely reduced by the adoption of the "stepped waste-weir" (paras. 188 to 195, pp. 260–280) when such a design is practicable. (Also, see Note 5 to Appendix 11, p. 369.)

The second curve for Bombay catchments is due to the late Mr. H. F. Beale, M.Inst.C.E. For areas less than 100 square miles it is below the third curve, but afterwards rises above it. It, however, then becomes so flat that it probably would not give large enough values for greatly increased catchments even if it sufficed for smaller ones.

The third curve for Bombay catchments was devised by the late Mr. J. E. Whiting, M.A., M.Inst.C.E. For areas of 25 square miles and under it does not differ greatly from the first curve, but for those above 100 square miles becomes extremely flat.

The Deccán catchments are subject to very heavy rainfall, and the run-off from them is very great owing to their steep slopes and large extent of impervious and barren land.

The curves for Ceylon (low country) and Transvaal (middle and high veld) compare with each other, but are very much below those for Bombay owing to the flatter slopes covered with vegetation of their catchments, and to the smaller amount of rainfall on them. The Transvaal is liable to storms of great intensity but of short duration and comparatively small extent; it has fairly steep catchments, but they are generally grass-clad. Ceylon has less violent

storms but they extend over large areas. The drainage areas of reservoir sites there are generally not steep, and are usually covered by forests, which greatly diminish the intensity of the floods from small catchments, but, compared with the Transvaal, increase it for large catchments as they prolong the period of run-off. For neither of these colonies have records been maintained for a sufficient time to enable perfectly reliable curves to be drawn for their high-flood discharges. On December 20th, 1911, the high-flood discharge of the Kanakarayan Aru, near the north of Ceylon, at the site of the dam of the Iranaimadu reservoir (Karachchi project), where the river has a catchment area of 227 square miles, was gauged as 50,000 cusecs, which is nearly double the amount which would be deduced from the table. The rainfall was, however, much above the normal and probably cyclonic; 20 inches in one day were gauged at one place in the neighbourhood, and over 10 inches in one day at each of five other near stations.

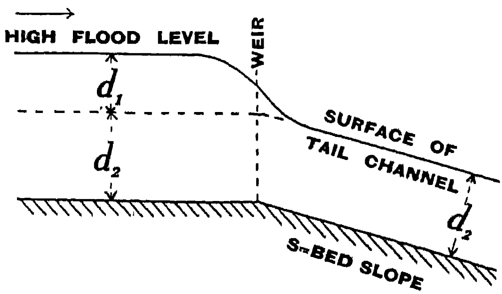
It will be noticed that all five curves illustrate the general law that the high-flood discharge from small catchments is relatively much greater than that from large ones having similar physical features. They all flatten considerably at their limit of 200 square miles, and this is one indication that, were they extended to include much larger areas, they would then become practically horizontal. In other words, after a certain limit had been reached, an increase of catchment area would not lead to an increase in the rate of flood discharge, as the run-off from the upper parts of the catchment would arrive at the point of discharge after that from its neighbourhood had diminished or stopped. This condition applies to

catchments having a sensibly uniform width and therefore affluents of much the same length. Where, however, a main river is joined by a considerable tributary, the condition would not hold directly but would be influenced by the effect of each of the catchments concerned.

In pervious formations when the rainfall decreases from the high lands at the heads of the catchments, streams have a tendency to decrease in volume as they proceed, and may indeed dry up towards their outfall. This occurs in the north of the Transvaal, where the "sand rivers" are generally waterless where they meet the Limpopo. In Jamaica, also, there is much loss in transit for the same reason.

172. Formulæ for the Discharge of Waste-Weir Channels.—For calculating the discharge of these channels (para. 160, p. 212) the following formulæ are used (Fig. 25):—

FIG. 25



$$\text{(i.)}\quad D = a\sqrt{r}\,.\,c_2\sqrt{s}\quad . \quad . \quad . \qquad \text{(Chézy)};$$

$$\text{(ii.)}\quad D = c_1 b\sqrt{2gd_1}\left(d_2 + \frac{2}{3}d_1\right)\quad . \quad \text{(Eytelwein)};$$

$$\text{or } \frac{D}{c_1 b\sqrt{2g}} = \sqrt{d_1}\left(d_2 + \frac{2}{3}d_1\right),$$

where D is the discharge in cubic feet per second;

a, the cross-sectional area of the tail flood in square feet;

r, its "hydraulic mean radius" =

$$\frac{\text{area (in square feet)}}{\text{wetted perimeter (in feet)}};$$

c_1, the afflux coefficient (Appx. 10, p. 366);

c_2, Bazin's coefficient for the tail channel (Appx. 9, p. 364);

s, the slope of the tail channel $= \dfrac{\text{fall}}{\text{length}}$;

b, the breadth, or length, of the weir in feet;

d_1, the height of the flood afflux in feet;

d_2, the depth of the tail channel discharge in feet; and

g, the force of gravity.

D and s being given, then a $(= b \times d_2)$ in the first formula is found by trial. From the second formula d_1 is ascertained, generally by trial to avoid a cubic equation. The height to which the flood will rise in the reservoir is $d_1 + d_2$, and this must be provided for, care being taken not to neglect d_1, as is sometimes erroneously done.

173. Calculation of the Discharge of a Waste-Weir Channel.—An example of the calculation of the discharge of a waste-weir channel is given below. The channel is assumed to be 200 feet wide, and to have vertical sides and a longitudinal slope from the crest line of 1 in 100, and the total flood depth to be 8 feet. In the first place it is necessary to assume both the tail depth and the afflux height, and then to make trial calculations until the correct results are obtained.

A. Tail Depth.

Assume $d_2 = 4{\cdot}48$ feet

$$D = a\sqrt{r}\,.\,c_2 z\sqrt{s}$$
$$= 896 \times 2{\cdot}07 \times 77{\cdot}5 \times 0{\cdot}1$$
$$= 14{,}374 \text{ cusecs}$$

$$a = bd_2 = 200 \times 4{\cdot}48 = 896$$
$$r = \frac{a}{wp} = \frac{896}{209} = 4{\cdot}29$$
$$\sqrt{r} \quad .. \quad .. = 2{\cdot}07$$
$$c_2 \quad .. \quad .. = 77{\cdot}5$$
$$s = 0{\cdot}01;\ \sqrt{s} = 0{\cdot}1$$

B. Afflux Height.

Assume $d_1 = 3{\cdot}52$ feet.

$$\frac{D}{c_1 b\sqrt{2g}} = \frac{14{,}374}{0{\cdot}7 \times 200 \times 8{\cdot}02} = \frac{14{,}374}{1{,}123} = 12{\cdot}80$$

$$\sqrt{d_1}\left(d_2 + \frac{2}{3}d_1\right) = \sqrt{3{\cdot}52}\left(4{\cdot}48 + \frac{2}{3}(3{\cdot}52)\right)$$
$$= 1{\cdot}88 \times 6{\cdot}83 = 12{\cdot}84$$

Therefore the two sides of equation (ii.) of paragraph 172 are approximately the same with the assumed values, and the assumptions are sufficiently correct. The total flood depth is :—

$$d_1 + d_2 = 3{\cdot}52 + 4{\cdot}48 = 8{\cdot}00 \text{ feet.}$$

In calculating $\sqrt{r}$ and $\sqrt{d_1}$ a table of square roots will be found most useful. Appendix 10, p. 366, gives the values of c_1, and Appendix 9, p. 364, those of c_2.

It may be noted, as an aid to adjusting d_1 and d_2, that for any given change of d_1 the value of the side of the equation $\sqrt{d_1}\,(d_2 + \frac{2}{3}d_1)$ will change less than will the side of the equation $\frac{D}{c_1 b\sqrt{2g}}$ for the corresponding variation of d_2 required to make the sum of $d_1 + d_2$ the same. This is illustrated by the following examples :—

1	2	3	4	5	6	7
Factors	Case 1.			Case 2.		
d_1	4 60	4 80	5 00	6 30	6 50	6 70
d_2	6·40	6 20	6·00	8·70	8 50	8·30
$d_1 + d_2$	11 00	11 00	11 00	15 00	15 00	15 00
$\sqrt{d_1}\left(d_2 + \frac{2}{3}d_1\right)$	20 27	20 59	20 90	32·38	32 84	33·07
$\frac{D}{c_1 b \sqrt{2g}}$	21 45	20 43	19 33	33 78	32 54	31 29

Columns 3 and 6 show the nearest correct approximations possible, taking only the first place of decimals into account for d_1 and d_2 and greater accuracy is not necessary in such calculations. (Compare Appendix 10 where the second place of decimals has been taken.)

174. The Discharge of a 200-foot Waste-Weir Channel.—A table of the discharges of a 200-foot waste-weir channel with a tail slope of 1 in 100, and with total flood depths varying, foot by foot, from 1 foot to 20 feet, is given in Appendix 10, p. 366. As the discharges vary with the extent of the flood's wetted perimeter, the discharges of weir channels with the same depth are not in exact proportion to their different widths. However, as the wetted perimeters of such floods very nearly vary with their widths, for all practical purposes the discharges of the floods may be taken as proportionate to their widths, and especially is this the case for the small depths which usually have to be considered. Examples bearing this out are given below :—

Width of channel . .	Feet	1,000	1,000	50	50
Total depth of flood .	,,	2	10	2	10
By direct calculation	Cusecs	7,322	106,555	362	5,180
Deduced from Appendix 10	,,	7,285	106,050	364	5,302

Appendix 10 will be found very useful in determining the width of the weir channel and the total flood depth to be adopted in any particular case.

Plate 2^B, which is due to Mr. J. A. Balfour, M.Inst. C.E., is based on Appendix 10, and by it either the afflux height d_1, or the tail depth d_2, for all ordinary total flood depths can be ascertained by inspection after the other has been calculated in accordance with what has been written above. It will be found most useful as it provides for variations in the slope of the bed and consequent differences in the velocity and depth of the tail channel and of the height of the afflux. To use this diagram that velocity will first have to be decided on with reference to what the ground forming the bed of the tail channel can stand without excessive erosion. The total discharge which has to be provided for being known, trial calculations[1] will have to be made with assumed tail channel bed-widths, depths, and slopes until the correct ones are ascertained, and thus the proper value of d_2 will be determined. The discharge of a 200-foot width of the tail channel will then have to be assumed to be in proportion to that of the total width calculated. By looking along the curves of the diagram the corresponding afflux height d_1 can at once be seen and the total flood depth $d_1 + d_2$ can then be found.

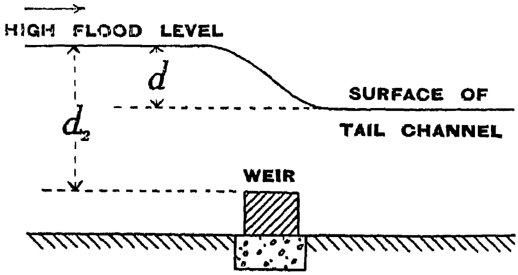
The diagram can be used in the reverse way by first assuming and then calculating the afflux height d_1, and from it ascertaining by means of the curves the tail depth d_2: the former method, will, however, generally be the simpler of the two. The results of Appendix 10 have been plotted on the diagram and are practically on a straight line.

175. Formula for the Discharge of Drowned Waste-

[1] Higham's "Hydraulic Tables" published by E. & F. N. Spon will facilitate the making of these trial calculations.

Weirs.—For calculating the discharge of these (para. 160, p. 212) the formula [1] used is (Fig. 26) :—

FIG. 26



$$\text{(iii.)}\ D = c_1 b \sqrt{2gd} \left(d_2 - \frac{d}{3}\right),$$

where D is the discharge in cubic feet per second ;

c_1, a coefficient, which may be taken as in Appendix 10, p. 366.

b, the breadth, or length, of the weir in feet ;

d, the height in feet of the surface of the afflux above that of the tail channel ;

d_2, the height in feet of the surface of the afflux above the weir crest ; and

g, the force of gravity (32·2 feet per second, Appendix 25 (III), p. 479).

The calculation of the discharge is not a straightforward one, and an example of it is not worked out. A table cannot be prepared for such weirs as their discharging capacity depends upon the depth of their crests ($d_2 - d$) below the surface of the tail channels, and this is variable. The depth of the tail channel has to be determined by means of formula (i.) of

[1] Prof. Unwin's article on Hydromechanics in the "Encyclopædia Britannica," Vol. 12, p. 473, 9th Edition.

paragraph 172, p. 236. The afflux height has to be calculated by assumption, and has therefore to be ascertained by trial calculations unless a cubic equation is worked out.

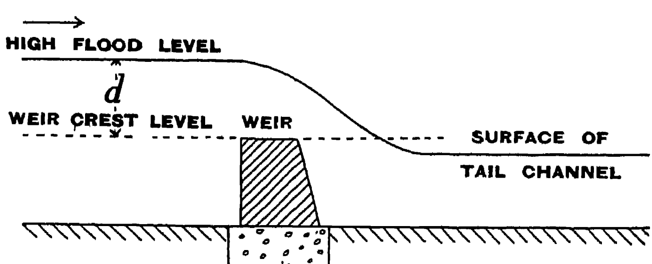
176. Formula for the Discharge of Clear Overfall Waste-Weirs.—For calculating the discharge of these (para. 161, p. 212), Francis' formula [1] (Fig. 27) is used, namely :—

$$\text{(iv.)}\ D = \frac{2}{3} cbd\sqrt{2gd} = 5{\cdot}35\, cbd^{\frac{3}{2}},$$

where D is the discharge in cubic feet per second ;
- c, the coefficient (Appx. 11, p. 369, note 3) ;
- b, the breadth or length of the weir in feet ;
- d, the height in feet of the afflux above the weir crest ; and
- g, the force of gravity (32·2 feet per second, Appx. 25 (III), p. 479).

The calculation of the discharge in this case is a straightforward one, and an example of it is therefore

FIG. 27



not worked out. A table of the discharges of clear overfall waste-weirs varying in depth by one-tenth of

[1] Prof. Unwin's Article on Hydromechanics in the "Encyclopædia Britannica," Vol 12, p. 472, 9th Edition.

a foot from zero to 10 feet has been calculated by this formula and is given in Appendix 11, p. 368.

In all these formulæ the surface of the afflux is that of the still-water level of the reservoir, or, in other words, the afflux surface is its high-flood level.

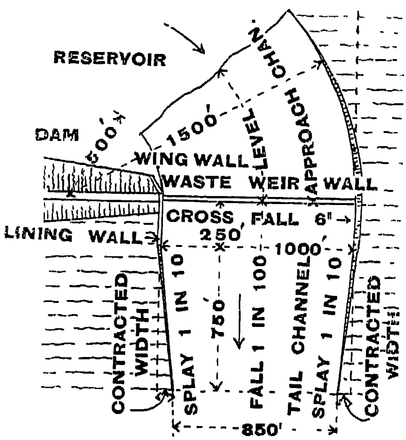
II. The Tail Channel.

177. The Section of the Channel.—Not only must the flood discharge have a clear approach to the waste-weir, but also it must have a perfectly unobstructed exit therefrom for some distance from the weir crest. Obstructions in that length will cause the flood to head up, and make it mask and lessen the discharging power of the weir unless the crest of that is higher than the surface of the tail channel. If, however, the contraction of the tail channel will not cause the flood to head-up above the weir crest itself, it will, of course, not interfere with the discharging capacity of the weir. The amount of excavation of a long tail channel can therefore be considerably reduced by gradually contracting its width as shown in Fig. 28. As in the case of the approach channel (para. 164, p. 218), the curves of the head of the tail channel should be ones of large radius.

Another advantage of thus contracting the channel is that the floods, instead of being distributed over a very wide bed, will be confined to a comparatively narrow cut, and will thus tend to keep to a defined course and not to spread over a large area, when they might perhaps excavate for themselves irregular scour channels, which might cause injury to culturable lands.

FIG. 28

PLAN



The channel should start from the waste-weir with the calculated bed fall. If this involves a large amount of excavation, the slope may be reduced as soon as such reduction can be effected without causing heading-up at the weir crest. Where, however, the natural fall of the country exceeds that of the calculated bed fall, the channel may be given as steep a slope as the former, provided that thereby excessive erosion is not caused up to the waste-weir itself. The tail channel should be led as soon as possible into a natural drainage line, so as to diminish the amount of excavation and so as to secure a regular course for the floods without a necessity for protective embankments. The flood water should be confined to the tail channel by means of lining walls, pitched slopes, or flood embankments, until it can no longer cause injury to the reservoir works or to cultivated lands, buildings, etc. (Plate 4, Fig. 1).

For flank weirs, in order to direct the flood away from the dam, it is desirable to give the tail channel a small cross-sectional fall—say of 6 inches—to the side remote from the dam. As a general rule in flank weirs the inclination of the surface of the ground will be towards the dam, and the effect of this on the flood

discharge should be counteracted by an artificial cross-sectional fall in the opposite direction given to the tail channel.

The above remarks apply to channel and to drowned weirs. For clear overfall weirs little or no excavation of the tail channel will usually be necessary, as the flood waters will find their own way down the natural depression which will generally exist below the centre of the weir. Also, in the case of saddle weirs, when they are remote from the dam, the floods from them cannot injure the works, and it is usually cheaper to pay compensation for the damage they may cause to the fields through which they pass than to construct embankments or to excavate channels to protect them. It may, however, be useful, even for them, to excavate a shallow channel so as to direct the first overflows; the floods will thus be led at but little expenditure to scour out a deep cut down to hard material, and thus may effectually confine themselves to a defined course.

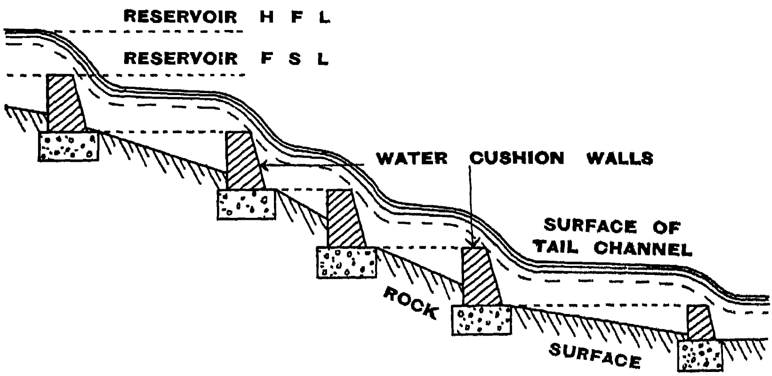
178. "Retrogression of Levels."—A tail channel does not usually widen itself, although severe floods passing down it before it has cut a regular course may cause the formation of scour channels branching from the main one. The effect of floods is generally to deepen the tail channel, and the deepening as a rule results from the cutting back of the bed where the soil is soft; the exposed face, even when of hard material, may thus be eroded away with rapidity: this action has been termed the "retrogression of levels". As long as the erosion does not extend to the weir, it does no harm, but if it threatens to reach that work, it must be prevented by the construction of curtain walls built right across the tail channel and founded securely,

and, if possible, in rock. Where hard material does not exist in the channel bed, these walls must be protected by pitched rapids, or, better still, by water-cushions, which can be formed by raising the crests of the lower walls.

This retrogression of levels is most likely to occur at a site where the waste-weir is situated on a ridge with a steep fall downstream, as even in the case of rock, all but the soundest unfissured variety is liable to be detached in large blocks by the rapid descent of floods

FIG. 29

SECTION OF WATER CUSHIONS IN SERIES



over it. The best treatment of such a site is to form a series of water-cushions down to the general level of the country, as each of them will destroy much of the horizontal velocity of the water, and the flood will pass with a comparatively gentle flow over the lowest one (Fig. 29). It will facilitate design and construction if all the water-cushion walls are made of the same height and section; their distances apart will thus vary with the slope of the ground.

The great advantage of curtain and water-cushion walls is that they distribute the floods evenly over the bed of the tail channel downstream of them and thus prevent the formation of scour channels, which might rapidly deepen with each flood, since the discharge will be concentrated down them.

179. The Effect of the Bed Slope on the Velocity of the Tail Channel Discharge.—The velocity of the flood down a tail channel is calculated by the formula (para. 172 (i.) p. 236):—

$$V = c_2 \sqrt{r\,s},$$

where V is the velocity in feet per second;

c_2, Bazin's coefficient (Appx. 9, p. 364);

r, the hydraulic mean radius =

$$\frac{\text{area (in square feet)}}{\text{wetted perimeter (in feet)}}; \text{ and}$$

s, the slope of the channel $= \dfrac{\text{fall}}{\text{length.}}$

The effect on $\sqrt{s}$ of increasing the slope is at once apparent from the figures given below:—

Bed Slope, 1 in	5,280	2,640	1,320	1,000	500	200	100
s =	0 000189	0·000379	0 000758	0·001	0·002	0·005	0 01
$\sqrt{s}$ =	0 014	0 019	0·028	0 032	0·045	0·070	0·100

These show that $\sqrt{s}$ is seven times as great when the slope is 1 in 100 as when it is 1 foot in a mile. An increase of s will, however, diminish r and c_2, but on the whole will increase V in the formula.

The slope to be selected depends chiefly upon the nature of the bed of the head of the tail channel, as the velocity produced by it should not exceed that which the soil can stand without excessive erosion. For total flood depths not exceeding 6 feet, where the

channel is in rocky ground a slope of 1 in 100 can be given as a maximum; where it is in hard ground, the slope should not exceed 1 in 500; and where it is in ordinary soil, it should not be steeper than 1 in 1,000.

The effect of reducing the inclination of the slope while maintaining the weir of the same length is, of course, by lessening the velocity, to increase the depth of the tail flood, and, thus in the case of channel and drowned weirs, to raise the afflux height, and to add to the cost of the dam which has to be constructed correspondingly higher. Where the tail slope has to be made flat, it will generally be found cheaper to lengthen the waste-weir rather than to raise the dam, but the decision as to what has to be done in each case should be arrived at after making alternative estimates of the two works combined. ' It will usually be cheaper by flattening the slope to reduce the velocity of the tail channel discharge than to construct protective works to control it. Too high a velocity may be dangerous, and too low a one may entail unnecessary expense.

Where a weir has its outfall into a stream parallel to itself and with a bed considerably below its crest level, the weir should, when the ground is soft, be placed as far away from the stream as is necessary in order to reduce the inclination of its tail channel; or, if this is not practicable, a good water-cushion should be formed below the weir. If, however, the tail channel is rocky and can withstand the overfall, the weir may be advanced towards the stream.

180. The Outfall of the Tail Channel.—The most desirable outfall for a waste-weir is one into the stream on which the reservoir is constructed, and as near to the

dam as is practicable without injuring the works or interfering with their drainage. This latter is a most important consideration, as the floods, when cutting out a new channel for themselves, will bring down a large amount of detritus, which will tend to choke the bed of the main stream and to block channels excavated in it for drainage.

Where the outfall is to another stream, unless the channel of that stream is ample to pass off the added drainage (which will seldom be the case in India, where heavy floods usually overflow the banks), damage may be caused to the neighbouring lands. The bed, too, of the original stream being deprived of scour, may silt up, thus interfering with the drainage of the dam; and may become marshy and covered with rank vegetation, thus leading to malaria. The subsoil water level of the riparian lands may be raised, and perhaps a saline efflorescence on them may be formed, thus rendering such fields unculturable.

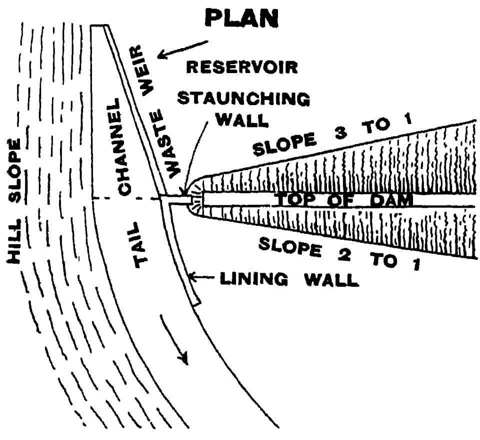
181. Design with the Tail Channel Parallel to the Waste-Weir.—In the previous paragraphs tail channels at right angles to the waste-weir have been discussed. Such channels are in general use, but another form is practicable in which the channel at starting is nearly parallel to the waste-weir; this is illustrated in Plate 4, Fig. 1, and Plate 6, Fig. 2. In the case of the Máládevi tank project the parallel channel was adopted to lessen excavation and to get the tail discharge in a defined channel suitable to the general design of the weir with its under-sluices and automatic gates.

This form is chiefly adaptable to sites where the length available for an ordinary waste-weir is restricted in extent by high ground, as is shown in Fig. 30, where a hill is seen coming close to the dam. The waste-weir

is therefore designed nearly parallel to the high-flood contour of the ground, and at a distance from it sufficient for the space required by the tail channel. It will generally be best in such a case to build the weir as a clear overfall one, but, where necessary, it may be of the channel form, the tail channel being then excavated as a trench parallel to the crest of the weir.

In calculating the sections of such a parallel tail channel, its water surface slope must first be determined, as the discharge depends upon this, and not upon the bed slope; the steeper this surface slope, the smaller will be the sections of the tail channel. The area required to enable the channel at any point to carry the discharge due to the length of the weir upstream of it can be calculated in the usual way. The channel will thus have to be increased in width gradually from the upstream to the downstream end of the weir, and should be continued therefrom with its maximum width until this can safely be reduced, as explained in paragraph 177, p. 243. In making the calculations, account must be taken of the losses of velocity due to the tail water at the weir having at once to change in direction through nearly a right

FIG. 30



angle and to the effects of eddying motion. It is believed no such form of tail channel has yet been constructed for a large Indian flood, and it is therefore advisable to make full provision for these losses when this design is selected.[1]

By combining this design with that of the "stepped waste-weir" (paras. 188–191, pp. 260–267), a very large reduction may be made in the length of the weir compared with that of an ordinary level flank weir. Thus many sites not sufficiently long for the latter may be utilised by adopting the former design. The suggested combination may therefore render feasible a project otherwise impracticable.

III. The Flood-Absorptive Capacity of Reservoirs.

182. The Regulating Power of Lakes.—During high floods the surface of a reservoir rises when the waste-weir at the lower levels is not able to discharge the water at the same rate as that at which it is entering the tank. When the incoming flood decreases so that the discharging capacity of the waste-weir at the level attained by the reservoir becomes greater than the rate of inflow, the storage surface is gradually lowered. The action of a reservoir is, therefore, to moderate the intensity of a natural flood by absorbing part of it as it arrives, and to discharge this part after that flood has diminished.

The property of flood-absorption and discharge-regulation possessed by large lakes is well known, but it is believed that, at present, advantage of it has not been taken in the design of reservoirs constructed in

[1] Since this was written, a waste-weir to this design has been proposed for the very large Daddi Tank, Gokák Canal Extension Project, Bombay.

India. The best known natural instances of it are the great lakes of North America and Central Africa and the Lake of Geneva. It is probable that the size of these lakes compared with that of their inflowing rivers is relatively larger than is the case with Indian reservoirs, and also that the maximum falls of rain filling them are of less intensity than those which occur in India, so that the rise of the storage during heavy storms is less there than it is in Indian tanks.

183. The Necessity for Large Absorptive Capacity.—To utilise this property of regulation sufficiently and safely it is essential that the storage capacity of the reservoir between its "restricted" (para. 184, p. 253) or full-supply level and its high-flood level should be large in comparison with the yield from the drainage area. Otherwise, a prolonged storm of great intensity may fill the tank to the last-named level, and, if it continues thereafter, there will not be any flood-regulation possible, and the waste-weir at high-flood level must, therefore, have a discharging capacity equal to the maximum rate of inflow. It is for this reason that in existing reservoirs advantage has not been taken of this property. It is seldom in them that the calculated high-flood depth over the weir exceeds 6 feet, that depth being fixed as a limit so as not to have a discharge of great volume per foot run over the weir and down the tail channel. There have been instances in Bombay of actual floods exceeding the calculated high-flood level when that was less than 6 feet, although, when the waste-weir discharge was calculated, the flood-absorption of the reservoir had not been taken into account. It would therefore appear advisable not to make an allowance for the influence of the reservoir in regulating the flood

discharge when the calculated high-flood depth over an ordinary weir is less than 6 feet.

It may, of course, be said that these instances of excessive flood prove that the rate of maximum run-off was under-estimated, as, during the rise of the reservoirs up to calculated high-flood level, the rate of inflow must greatly have exceeded the calculated maximum discharging power of the waste-weirs. The flood-absorption during this period may, however, be considered to provide a factor of safety for the calculated discharging capacity of the weir.

184. Method of safely utilising the Property of Flood-Absorption.—As it is not prudent to increase the depth of the maximum flood over a waste-weir, another plan must be adopted if the flood-absorptive property of the reservoir is to be utilised. This plan consists in tapping the reservoir at a low level, so as to "restrict" it to such level before a heavy flood discharge arrives, so that that flood thereafter will have to raise the reservoir surface to a considerable extent; while the surface is thus being raised the tank will absorb a large part of the run-off. During this period the rate of outflow from the reservoir will be less than the actual high-flood discharge of the river, and the natural intense flood of short duration will then be converted into a prolonged one of less volume. The action on the waste-weir tail channel of the long-continued, gentle discharge will be less than that of the short, intense flood.

It must be remembered that the rise of the reservoir surface during a flood is very much more gradual than that of the inflowing stream, and that the maximum discharge of that stream is produced long after the rain has commenced to fall. There is thus ample

time from the beginning of a heavy storm to lower the tank to a considerable extent if sufficient provision has been made in the design of the waste-weir to permit of the escape of a large discharge at a low level. A design which will thus safely utilise the flood-absorptive capacity of the reservoir is described in paragraphs 188 and 191, pp. 260 and 269.

Not only must a waste-weir be able to deal with an individual flood, but it must also be capable of disposing of a second flood following the previous one at a short interval. The calculations of discharge made in Appendices 12 and 14, pp. 370–375, taken in the reverse order to that in which they are therein entered, show that a waste-weir designed on the lines recommended, will soon lower the surface of the reservoir after a flood, and will thus enable the work quickly to regain flood-absorptive capacity so as to dispose safely of a subsequent flood.

185. The Allowance for Flood-Absorption in the Reservoir.—In order that the provision for flood-absorption in the reservoir may be sufficient to prevent its calculated high-flood level from being exceeded, full allowance must be made for the total yield from the maximum flood which may be expected from the catchment. Sir Thomas Higham has discussed this subject in a note, dated 24th March, 1902, on tank projects and the method of dealing with waste-weirs in the Central Provinces. He lays down the following rules in the two cases considered :—

(*a*) *When the Waste-Weir alone is taken into account.*

" Unless the absorptive capacity of the tank above crest level is greater than half the total influx during the continuance of the flood (which is not ordinarily

the case), the waste-weir must be designed to pass the whole run-off per second, and the flood-absorbing capacity of the tank above crest-level must be neglected."

(*b*) *When the combined effects of the Waste-Weir and Outlet are taken into account.*

"The absorbing capacity of the tank must be neglected unless this will be greater than half the total inflow during the continuance of the flood less half the amount that would pass through the outlets during the same time, assuming the rate of outflow to be that obtaining at commencement of flood, or when the water surface in the tank is at crest level."

These rules have been based on mathematical considerations, but, as the intensity and duration of a maximum storm cannot be predicted, the following rules based on practical considerations are suggested :—

(*c*) *Rules for the Provision of the proper extent of Reservoir Flood-Absorptive Capacity.*

(i.) During the rise of the water surface in the reservoir, the flood disposed of hourly (by absorption in the tank and by the discharge from the waste-weir and the outlet) should at least be equal to half the calculated maximum rate of run-off from the catchment.

(ii.) During the total rise of the water surface in the reservoir to calculated high-flood level, the total flood disposed of should at least be equal to the yield of the calculated maximum flood from the catchment.

(iii.) The combined discharging capacity of the waste-weir and of the outlet at high-flood level should at least be equal to half the calculated maximum rate of run-off from the catchment.

It will be noticed that these rules (*c*) provide for the effect of the capacity of the reservoir relative to the size of the catchment and also take into account the anticipated rate of run-off and the yield from it during heavy rainfall. They also provide a considerable margin of safety for abnormal floods by implying that the difference between the "restricted" level of the tank at the commencement of the flood and the calculated high-flood level shall be a large one.

185A. Mathematical Calculation of the Effect of Flood-Absorption.—The question how far it is desirable to reduce the length of the waste-weir on account of the flood-absorptive capacity of the reservoir has been exhaustively discussed from a mathematical point of view by the late Major A. ff. Garrett, R.E., in his pamphlet, "The General Theory of the Storage Capacity and Flood Regulation of Reservoirs" which was published at Calcutta in 1912, by the Superintendent, Government Printing, India. He has taken into account the flood results of numerous tanks, chiefly in the Central Provinces, where, however, the record extended only to five years (which is too short), and has devised formulæ, and based tables thereon, for calculating the length of waste-weir required when the flood-absorptive capacity of the reservoir is allowed for. He states that these formulæ give results closely in accord with observations made and provide a margin of safety; also that by adopting the formulæ the lengths of waste-weirs have been reduced considerably—in certain cases by more than 50 per cent.—when compared with the lengths usually allowed. (As pointed out in paragraph 183, p. 252 above, the fact, however, remains that abnormal floods (the ones to be provided for) have been known

to exceed the designed high-flood levels (when their heights above full-supply level were comparatively small), although the flood-absorptive capacity of the reservoirs was not considered when the lengths of the waste-weirs were calculated.)

The author admits in his preface that where the difference between the full-supply and high-flood levels is great the problem is difficult to deal with mathematically, and, as far as he can see, can be solved only by trial and error. (In paragraph 183 above it is stated it is only in such cases that advantage should be taken of the flood-absorptive capacities of reservoirs.) He further allows, in his paragraph 29, that the efficiency of his formulæ, "as in the case of all results based on mathematical analysis, depends entirely on the soundness of the assumptions made—in the present instance on the value adopted for the maximum flood influx i, and its period of duration, T." In the same paragraph he adds, "local conditions are so variable that it must always remain impossible to fix runs-off and durations of floods with any great degree of accuracy, and the figures which will prove most suitable must therefore be very largely a matter of judgment and experience, based on knowledge of the peculiarities of the catchment and local rainfall." (These conclusions appear eminently sound, and greatly to minimise the value of the mathematical treatment of the subject.)

186. Examples of Reservoir Flood-Absorption Calculations.—The flood calculations of the Máládevi tank project are selected as examples of calculations on the basis of rules (c), p. 255. That tank has a catchment of 153 square miles, for which, according to Appendix 8, p. 363, the average rate of run-off should be 0·80 inch

per hour. As the drainage area is a good one for producing discharge, the rate of run-off might, however, be taken as 1·00 inch per hour.

Appendix 12, p. 370, gives a calculation for the temporary waste-weir, first closure. During the rise of the tank, the run-off is disposed of at a rate varying from 0·44 to 0·68 inch per hour, and the total run-off dealt with in this period is 4·75 inches. The high-flood discharging capacity of the weir is at the rate of 0·546 inch per hour.

Appendix 13, p. 372, gives a calculation for the temporary waste-weir, second closure. During the rise of the tank, the run-off is disposed of at a rate varying from 0·44 to 0·62 inch per hour, and the total run-off dealt with in this period is 3·98 inches. The high-flood discharging capacity of the weir is at the rate of 0·42 inch per hour, which is somewhat small, but the work as it would be constructed (see the note to the Appendix) would have a greater discharge.

Appendix 14, p. 374, gives a calculation for the permanent waste-weir. During the rise of the tank, the run-off is disposed of at a rate varying from 0·44 to 0·75 inch per hour, and the total run-off dealt with in this period is 6·96 inches. The high-flood discharging capacity of the weir is at the rate of 0·418 inch per hour, and to this has to be added the discharge of the outlet, at the rate of 0·07 inch per hour (para. 200, p. 286), giving a total high-flood discharging capacity of 0·488 inch per hour.

IV. The "Stepped Waste-Weir."

187. Objections to the Usual Form of Level Waste-Weir.—The usual form of a solid weir wall with a permanent crest raised throughout to full-supply level

is best suited for reservoirs the replenishment of which is not certain, as for them the storage level has to be kept as high as practicable during the whole of the monsoon, so that full storage may, if possible, be obtained at the end of the rains.

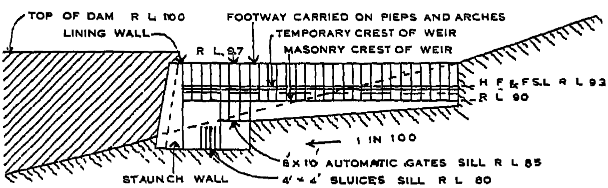
The great merit attributed to such a form of weir is that it is considered to be perfectly automatic in its action. However, the weir will not be automatic if its length has been under-estimated for ordinary floods, or if it has to deal with abnormal ones. Still so much has the value of this property of automatism impressed many engineers that they consider any form of non-automatic weir unsound in principle, although the working of most engineering schemes requires human agency for their management.

The principal objections to a solid and level weir are that it tends to keep the reservoir full during that part of the year when it is most difficult to effect repairs; it prevents the reservoir from being lowered rapidly when necessary in the case of an accident to the dam; it increases the action of the overfall on the foundations; it impounds the earliest monsoon floods, which are always the most heavily charged with silt (para. 36, p. 57); and, by maintaining the largest reservoir capacities, it gives all floods passing through the tank the maximum time in which to deposit their silt. Further, with it the flood-absorptive capacity of the reservoir cannot be brought fully into play and hence the length of the weir may have to be doubled beyond what is necessary when that capacity can properly be utilised. Finally, the storage capacity between full-supply and high-flood levels cannot, as a rule, be made safely available with this form of weir.

188. General Description of the "Stepped Waste-Weir."—To meet the objections to a solid weir with a permanent crest, the "stepped waste-weir" design is put forward. A form adapted to a saddle site is illustrated in Plate 6, and one suitable for a flank site is sketched in Fig. 31. For the latter it will be best to place the under-sluices and automatic gates some distance away from the end of the dam, so as to avoid the formation of a deep channel at this side, although the position thus selected may entail an increase of excavation in the tail channel.

FIG. 31

LONGITUDINAL SECTION



Such a weir may consist of five sections :—

(*a*) A drowned channel or weir ;

(*b*) A clear overfall weir ;

(*c*) An under-sluice section ;

(*d*) An automatic gate section ; and

(*e*) A temporary weir crest.

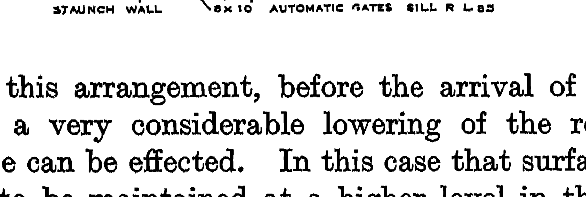
The levels at which the discharge will be passed through, or over, the work will thus vary considerably, and hence the name "stepped" given to this form of weir. By this variation of level, conformity with the natural profile of the ground can best be attained, for it will be but seldom that its surface will be level throughout the long length required for a weir ; hence,

with this kind of weir there may be a considerably lessened amount of excavation in the tail channel.

The stepped weir is best adapted to tanks with assured replenishments, for with them it is not necessary to arrange for securing the complete storage until near the close of the monsoon. It can, however, be used for tanks having uncertain replenishments, but for these it will be advisable to have a greater number of under-sluices placed with their sills at the lowest level practicable and having a maximum discharging power equal to that of the weir crest proper (Fig. 32).

FIG. 32

LONGITUDINAL SECTION



By this arrangement, before the arrival of a large flood, a very considerable lowering of the reservoir surface can be effected. In this case that surface may have to be maintained at a higher level in the early part of the monsoon than will be necessary for a tank with a certain replenishment.

189. Advantages of the "Stepped Waste-Weir."—This form of weir meets the objections enumerated against the level weir with a solid crest (para. 187, p. 259), and has the following advantages over that type.

(*a*) It enables the level of the reservoir to be "restricted" or kept low, during the early part of the monsoon, when the rainfall is most continuous and repairs can be effected only with difficulty;

(*b*) It allows the reservoir surface to be lowered rapidly when this becomes necessary in the event of an accident to the dam (a most useful advantage);

(*c*) It permits the earliest monsoon floods, which are those most heavily charged with silt (para. 36, p. 57), to be passed out of the tank in the shortest possible time and with the minimum deposit of silt;

(*d*) By keeping the storage of the reservoir as small as possible during the earlier part of the rains, it enables all floods during this period to pass out of the tank quickly and before they deposit all their suspended silt;

(*e*) It permits larger catchments to be utilised without fear of excessive silting;

(*f*) It brings the flood-absorptive capacity of the reservoir fully into play, and allows the capacity between full-supply and high-flood level to be stored for irrigation at the close of the monsoon;

(*g*) It enables the length of the weir to be reduced greatly (a matter of considerable importance for a site of restricted length);

(*h*) It allows a great variety of sites to be adopted for the waste-weir, as it is better suited to the usual natural profile of the ground;

(*j*) It converts a short flood of maximum intensity into a prolonged one of smaller amount, which will less injuriously affect the tail channel;

(*k*) It enables the tail flood to be directed along a defined and comparatively narrow channel, and thus may avoid the cost of lengthy protective works;

(*l*) The deepest part of the weir may be utilised as an outlet for a reservoir with a comparatively small depth of utilisable storage.

It may be added that the stepped waste-weir is adapted only to countries where the rainfall is confined to certain periods of the year and where storms of great intensity have to be dealt with: these conditions are, however, the usual ones in the tropics.

190. Objections to the "Stepped Waste-Weir."—The objections which may be raised to the stepped waste-weir are two:—

(*a*) It is not automatic;

(*b*) It may not be possible to utilise it owing to the uncertainty of securing full storage at the end of the monsoon.

In respect to (*a*) it may be said that throughout the early part of the monsoon, when the under-sluices and temporary weir crest would be fully open and the automatic gates arranged to open at once, the weir would be automatic, *i.e.*, it would pass the maximum flood possible without the need of any regulation by human agency. It is only at the close of the rains, when the final amount of storage is being, or has been, obtained by the closure of the under-sluices and by the erection of the temporary crest, that the weir will not be automatic. The reservoir surface will, anyhow, fall from high-flood level to full-supply level early in the cold weather (during which season storms of any intensity are hardly known in India), and thus it will be only for a few weeks at the end of and after the monsoon that the weir will have to be intelligently worked.

It must be remembered that the rise of a reservoir, even during a heavy flood, is so slow that it will never

be necessary to act with extreme haste. This type of weir is therefore much safer to work than a railway, where an error in signalling of a few seconds' duration may lead to a serious accident, or than aviation, where a defect of the machine may rapidly cause complete collapse. Risks have to be taken in many occupations—boilers and machinery with potential highly-destructive capacity and electric installations with death-dealing power are safely worked, and steamers in great numbers plough the seas although they have to face the numerous perils of the deep. Despite this ever-present liability to disaster, failures seldom occur: if their contingency were deterrent, progress would be stopped and civilisation would be at a standstill. With these examples before him the irrigation engineer must realise that he alone should not be progressive in designing his works. The fact is accidents are not likely to take place when their possibility is foreseen and guarded against by necessary precautions and watchful care; they are much more probable when there is a false sense of security which develops into carelessness. If skill can be relied upon to attain safety in the numerous dangerous occupations of man, it may equally be applied with confidence to designing and working the stepped waste-weir.

The objection (*b*) to this form of weir is more reasonable. There will certainly be some difficulty with a catchment having an uncertain yield to determine for how long the reservoir surface should be kept at, or below, the level of the sills of the lowest sluices without endangering the final loss of storage. The maintenance of the tank at this level is, however, chiefly required in the interest of diminishing its rate of silting, and for such storages that may be more

or less sacrificed. The principal utility of the stepped waste-weir for such catchments lies in its power of enabling the full-supply level of the reservoir to be raised to high-flood level at the end of the monsoon. This increase of storage can safely be effected by increasing the discharging power of the weir at its lowest levels (end of paragraph 188, p. 261, and Fig. 32).

With reservoirs having unfailing replenishments this objection has, however, but little force. Their catchments will almost invariably produce a sufficient amount of run-off at the close of the rains to complete the filling of the tanks. The best catchments will be those which furnish a fair weather discharge after the end of the rains of an amount sufficient to ensure the high-flood level storage, as thus a large proportion of the contents of the reservoir will consist of water having originally the minimum of silt suspended in it. An example of such a treatment is furnished by the Assuán[1] dam across the Nile.

191. Detailed Description of a "Stepped Waste-Weir."—The stepped waste-weir proposed in the Máládevi tank project is illustrated in Plates 4, 6, 7, and 8, and may be taken as an example of the general form of this type of weir.

Between the flanks the weir is designed with a clear overfall crest 1,004 feet long (with a net length of ventage between the piers on the crest of 800 feet), and having a maximum flood depth over it of 4 feet. At the centre are twenty 6 feet by 4 feet under-sluices worked by lifting screws and capstans, and protected against injury from floating logs, etc., by gratings formed of iron rails. Next to these sluices are eight

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[1] "Minutes of Proceedings, Inst. C E.," Vol. clii, Paper No. 3361.

8 feet by 10 feet automatic gates. Above all, and extending for the whole length—1,133 feet—of the weir crest, is an arcade carrying a foot-path and a tramway on which travelling winches can run: these will work the 10 feet by 4 feet teak shutters, which are used to raise the weir crest temporarily and thus to increase the reservoir full-supply level by 4 feet to high-flood level. It may be noted that 4 feet is probably the proper limit of height for such a temporary crest, and that it might be better not to make it more than 3 feet in height: in the present instance, owing to the short length available for the waste-weir, this could not be done except by a considerable increase of the under-sluices. At the extreme flanks are small embankments connecting the weir with the hill slopes: these have their tops lower than that of the main dam and are of smaller section than its crest, so that they may act as breaching sections (para. 75, p. 109) should any abnormal flood occur.

On the upstream side the high ground at the approach channel is excavated to the level of the masonry crest of the weir. On the downstream side are excavated cross channels, parallel and next to the weir (para. 181, p. 249), so that the surface of their high-flood discharge may be at an inclination of 1 in 100, and of gradually increasing width to enable them to pass off that discharge. To reduce the depth of the foundations of the weir and to protect them, curtain walls are placed at intervals across these channels. The lowest two of these walls (one at each side of the under-sluice section) form with it and the "water-cushion wall" a water-cushion, enclosed on all four sides by masonry, in which the whole discharge from the weir is collected and passed

down regularly to the tail channel. The sides of the cross channels opposite to the weir are protected from erosion by masonry lining walls, and small pitched flood embankments are carried parallel to the cross channels and to the tail channel, as far as this protection is necessary, to prevent the weir floods from flowing over the ground beyond them. The tail channel has at first a bed fall of 1 in 100, and afterwards one of 1 in 50, until it meets a natural deep watercourse into which the floods are turned. The width of the tail channel is gradually reduced so as to save excavation, care being taken that no heading-up and consequent retardation of discharge is thereby produced at the weir itself. By these arrangements the floods are tapped at a low level, and are passed down a defined channel to the river.

192. The Under-sluices and Temporary Weir Crest.—(*a*) *Under-sluices.*—The under-sluice gates (Plate 11, and Appx. 18A, No. 27) are of the ordinary cast-iron pattern, with planed gun-metal faces; these slide on cast-iron frames, similarly faced, and between cast-iron guides. They are raised by lifting rods which pass through the arcade piers so as not to be injured by floods flowing over the weir crest.

(*b*) *The Temporary Weir Crest.*—Probably the best arrangement for this will be that shown on Plates 7 and 8, and described in paragraph 191, p. 265. It entails the construction of an arcade on top of the weir, but it is advisable anyhow to go to this expense so as at any time to secure the means of dealing with any part of the weir and to gain access to the whole work.

On the large weirs of Northern India there are several ingenious forms of crest shutters, but, owing to

the great length of the works, they are of a somewhat cheap description, and it is difficult and takes time to close them, so that these designs do not appear suitable for the crest of waste-weirs which have a comparatively short length and a small crest width. However, such devices up to 6 feet in height are worked, and, if economy is absolutely necessary, they might be used on waste-weir crests, but their height had better be limited to 3 feet. For this, or a smaller, height some form of drum weir should be useful, as it might be designed in sections, say, 200 feet long, so as to be worked by water pressure from the ends of the weir. The Khavregat automatic shutter (para. 192B, p. 273) seems well adapted to such a weir crest.

192A. The Automatic Gates.—The automatic gates proposed are similar to those erected at the Bhátgarh reservoir, in the Poona district, Bombay Presidency, which are of the design patented by the late Mr. E. K. Reinold. These have been tested by actual experience for some years, and have proved quite satisfactory. The principal objection to them is that the gates travel downwards to open, so that they are not adaptable to all situations. Their action may thus be described (Fig. 33 [1]) :—

The gate, which may be 10 feet long by 8 feet high, travels downwards from its closed to its open position by means of rollers running on rails. To secure freedom of motion, the sluice frame is set at a very small angle to the gate, so that the two are in water-tight connection only when the gate is fully

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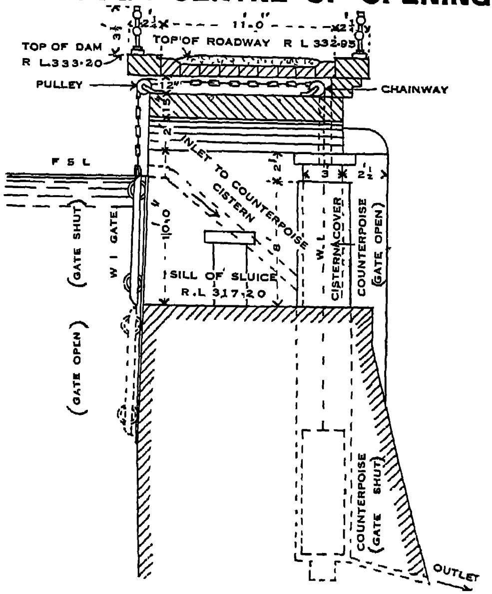
[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 196. *The Engineer*, November 3rd, 1893, pp. 430, 431.

raised and thus closed. Therefore, only the axle friction of the rollers and not the sliding friction on the frame has to be overcome. Suspended from the gate by two chains, one on each side, is a counterpoise weight, which works in a water-tight chamber, and has a motion reverse from that of the gate. At the calculated high-flood level there is a large inlet pipe leading from the reservoir to this chamber, and at the bottom of the chamber is a small outlet pipe discharging into the air. When the reservoir is at high-flood level the gate is at its maximum height, and then entirely closes the opening between the piers of the sluice-way. Should the reservoir surface rise to a higher level, water will find its way down the large inlet pipe and fill the counterpoise chamber, thus making the counterpoise lose weight, so that it is no longer able to keep the gate in its closed position. The gate then falls, and water is discharged through its sluiceway until the reservoir surface falls below the mouth of the inlet

FIG. 33

CROSS SECTION

THROUGH CENTRE OF OPENING



pipe. When this happens the outlet pipe drains the chamber, the counterpoise regains its full weight, and, descending, raises the gate to its closed position. This action of the counterpoise can be called into play at any designed level by fixing the mouth of the inlet pipe accordingly, or, by means of a valve worked by hand which will admit water through a pipe to the chamber when desired.

Several other forms of automatic gates have been recently patented, but as yet not much practical experience of their working has been gained. Some have failed as they get clogged with *débris*, which, for this reason, should be prevented from entering the gate arrangement. To ensure that the action will be automatic, the design must be simple, and without anything which may jam or become obstructed; to obviate the latter defect the vents should be comparatively large.

By a small modification of Mr. Reinold's design the counterpoise could be made to descend and the gate to rise when the reservoir surface exceeded high-flood level. This might be done by making the counterpoise hollow and larger, and by leading into it a pipe at high-flood level to fill and load it, and by having a smaller pipe at the bottom of the counterpoise to drain it: the counterpoise chamber would, of course, have to be kept free of water. If on account of the levels of the ground and water being high the counterpoise has to be fixed at a high level, it might be filled from an upper cistern, supplied by a hydraulic ram, and the discharge of the pipe connecting the two could be controlled by a valve actuated by a float which could be made to come into action at the desired limit of height of the reservoir surface. The arrangement of

the gate to lift will permit it to be used on a river weir, the bed of which is likely to silt up, or on a low weir of a reservoir, and will also obviate the initial overfall action of the flood on the crest of the weir and lessen that on its foundation.

On the waste-weir of Lake Fife, near Poona, have been erected automatic gates, somewhat similar to Mr. Reinold's, which have been patented by Mr. Visvesvarya, C.I.E., M.Inst.C.E.[1] These gates work in pairs, and are hung together by chains: the gates of each pair are of different weights, so that the counterpoise to which they are attached in common has to deal only with the difference of their weights and can thus be made small. The counterpoise is weighted so that when its cistern is empty its own weight is sufficient to pull up, or shut, the heavier gate, and then the lighter gate falls by its own weight and also closes its sluice vent. When the cistern is filled with water, the counterpoise loses weight, the heavier gate then falls and draws up the lighter one, and the vents of both are thus opened. The action of the counterpoise can be controlled by a valve as in the case of the Reinold pattern.

Mr. Visvesvarya has also patented another form in which all the gates are of equal weight and rise to bring the waste-weir into action. Each gate is connected by a chain to an upper balance weight working in a chamber constructed in a sluice pier, which weight is not quite sufficient to overcome the combined resistance of the gate and its counterpoise in the dry to be lifted so as to open the sluice-way. The gate is also attached to a counterpoise working in a cistern as before; the

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[1] Buckley's "The Irrigation Works of India," 2nd edn., p. 200

weight of this in the dry, *plus* that of the gate itself, is sufficient to cause the gate to fall and close its sluice opening. When water is admitted into the counterpoise cistern the counterpoise loses weight, and the balance weight draws up the gate and opens the sluice.

The Reinold gate transmits the whole of the water-pressure on to the axles of only four rollers, which may thus tend to become distorted, or the rollers may get stiff by rust or cut by grit. The well-known Stoney sluice [1] (which, however, is non-automatic) avoids these defects by transmitting that pressure to numerous free rollers suspended in a frame : these rollers bear directly on to the fixed frame of the sluice and thus enable the gate, even when of large size, to work easily even under great heads. The planed face of the gate is on its up-stream side, so that the tendency of the water pressure is to separate the gate from its frame during its upward or downward travel, and the two come into water-tight connection only when the gate is fully closed. This form of sluice has, it is believed, not been used in connexion with an earthen dam, but could with advantage be adopted for the under-sluices of the stepped waste-weir so as greatly to increase their size, discharging power and rate of manipulation, and consequently their efficiency.

As the automatic gates have to be placed near the crest level of the weir they have not the same rate of discharging power as the under-sluices, which can be fixed at any required lower level. Were a careful watch kept over the reservoir surface when it is at, or near, high-flood level (and it should always be easy to arrange for this at a work of any importance), automatic gates

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[1] "Minutes of Proceedings, Inst. C.E.," Vol. lx., p. 88.

would not be a necessity, and might be replaced by an additional number of under-sluices. In any circumstances, it will be better to provide for a larger discharging capacity by under-sluices than by automatic gates.

192B.—The Kharegat Automatic Shutter.—This shutter, which was designed and patented by Mr. M. R. Kharegat, Assoc.M.Inst.C.E., resembles, but is superior to, a drum weir in that it is automatic and does not require regulation by valves, and its moving parts are simple and not likely to clog or jam. It seems well adapted for forming the temporary crest of the waste-weir of a reservoir, and with it that crest might be increased to a height of 4 feet. It is illustrated in Fig. 33B and described below.

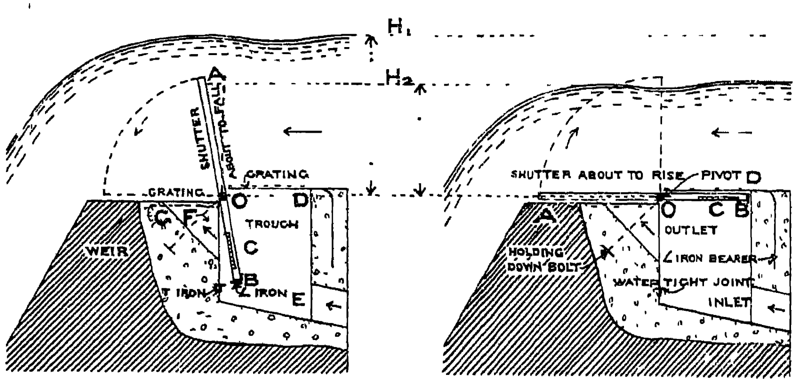
The shutter AOB is made of mild steel, as light as is consistent with sufficient stiffness, and is pivoted at O to the crest of the weir; each pivot is fixed to the masonry by ragged bolts for small shutters and by ordinary bolts with anchor plates for large ones. The upper portion, AO, of the shutter works sometimes in air and sometimes in water: the lower portion, OB, is always submerged when water is at or above crest level as it is placed in a trough; to it is fixed a counterweight, C, to assist its action. Water is admitted to the trough (which is a rectangular chamber) by the inlet, E, which can be closed by a small perforated movable shutter, and the trough is protected from the entry of rubbish by a grating fixed as a cover on top of it. At F the weir crest between the pivots of the shutter is chamfered off so as to let water pass up from the trough. A shutter is usually made about 10 feet long and its height from 6 feet to $1\frac{1}{2}$ feet for use on a river weir, in which case the shutters abut on each other

to form a continuous movable crest on the weir. For a reservoir waste-weir with an overhead arcade, each bay of that can have a shutter fitted to it separately. The pre-War cost of the shutters was about Rs. 20 per square foot.

FIG. 33B

KHAREGAT AUTOMATIC SHUTTER

SHUTTER FALLING SHUTTER RISING



The arrangement works thus. When the water level rises to a certain height, H_1, the shutter is over-balanced and falls, the portion AO oscillating near the crest and the portion OB being housed under the grating OD covering the trough: the weir crest is then unobstructed for the passage of floods. When the water level falls to a certain height, H_2, the shutter commences to rise by the flow of water up F under OA, which motion is assisted by the effect of the counter-weight C. In its final position the shutter is vertical and the counterweight makes OB close the inlet end of F. The rise of the upper part of the shutter is a

gentle one, following upon the fall of the surface of the reservoir, and thus storage is impounded early: the water in the trough acts as a brake steadying the motion of the lower part of the shutter. The levels H_1 and H_2, at which these actions begin to take place, depend upon the weight of the counterweight and the ratio to each other of the two portions AO and OB of the shutter which has to be adjusted so as to obtain full storage.

193. Flood Calculation of the "Stepped Waste-Weir." —Appendix 14, p. 374, gives a calculation showing how the stepped waste-weir of the Máládevi project can dispose of an extreme flood by discharging part of it and by allowing the balance to be absorbed by the reservoir. Starting with a discharge of 7,600 cubic feet per second (which is equal to a run-off of 1·85 inches a day from the catchment, and is a fair small flood), it will be seen that in 13 hours a total flood equivalent to a run-off of 6·96 inches is disposed of. Of this 1·75 inches are passed off by the weir, and the balance, 5·21 inches, is absorbed by the reservoir. When high-flood level is reached, the discharge of the weir crest and of all the sluices and gates will be at the rate of 0·418 inch an hour run-off, and assuming the reservoir remains at this level, the total run-off during the day will be 11·56 inches. These runs-off are far in excess of what may be expected from the catchment, and the waste-weir provision is thus shown to be ample.

The calculated high-flood discharge of the sections of the weir is :—

	Cubic feet per second.
(*a*) Clear overfall weir crest, 800 feet in net length and 4 feet deep	22,827 [1]
(*b*) Twenty under-sluices, each 6 feet by 4 feet	12,000
(*c*) Eight automatic gates, each 8 feet by 10 feet	6,457
Total	41,284 [2]

This is equal to a run-off of 0·418 inch an hour from the catchment, and the flood calculation shows that this provision is ample, although a discharging capacity of 1·00 inch per hour would probably have been required had a level waste-weir been adopted.

In addition to the above high-flood discharge, that of the outlet under-sluices (para. 200, p. 286) is 7,027 cubic feet per second, or at the rate of 0·07 inch an hour run-off from the catchment.

194. Working of the "Stepped Waste-Weir."—The arrangements proposed in the Máládevi tank project may be described as an example of how a stepped waste-weir may be worked in the case of a reservoir with an unfailing catchment.

During the early part of the monsoon the outlet sluices would be kept fully open, and, owing to the flood-absorptive property of the reservoir, it would only be rarely that the waste-weir would then come into action. By the beginning of September it would probably be necessary to commence to effect storage above the sill

[1] This was calculated from the table in Molesworth's pocket book; as calculated from Appendix 11, p. 368, the discharge is 21,728 cubic feet per second End contractions have not been allowed for as the piers on the weir crest will have pointed ends.

[2] The waste-weirs at both sides of the Bohio Dam, Panama Canal, are calculated to discharge together 1,570 cubic yards (42,390 cubic feet) per second. "Minutes of Proceedings, Inst. C.E.," Vol. cxliv., Paper No. 3207, p. 164.

of the under-sluices—R.L. 177·00—at which level the reservoir contents are 2,202·486 million cubic feet, or 0·43 of the high-flood level storage (Appx. 17, p. 382). By the beginning of October the reservoir would be allowed to fill to the masonry crest of the weir—R.L. 196·00—at which level its contents are 4,498·935 million cubic feet, or 0·88 of the high-flood level storage. During October the balance—613·418 million cubic feet, or 0·12 of the high-flood level storage at R.L. 200·00—would have to be stored. The increments of storage represent, respectively, runs-off from the catchment of 6·20, 6·46, and 1·72 inches.

It will be seen from this that up till September the outlet sluices and the waste-weir sluices and crest would be kept quite open and unobstructed, and the reservoir would thus be far safer than one with an ordinary level waste-weir. During September and October the rise in reservoir surface level would have to be carefully regulated. In November the reservoir surface would be maintained at high-flood level, as far as draw-off and replenishment would permit. By the end of that month it would probably have fallen below that level, and the reservoir would thus gain flood-absorptive capacity, which would rapidly increase as the season advanced, until, by the end of December, the masonry weir crest would probably be above water-level, and then the temporary crest could be removed.

The weir would thus require careful supervision by a superior staff only during September and October, as thereafter in this locality there is no fear of storms. Such a staff would be wanted for the working of the whole irrigation project, and could easily devote its especial attention to the reservoir during this period.

195. Saving effected by the Adoption of the "Stepped Waste-Weir."—It has been explained in paragraph 166, p. 220, that the provision of the margin of safety between full-supply level and high-flood level involves what may be considered either as a very expensive, but essential, addition to the cost of the dam, or as a surrendering of a large amount of storage. This margin is absolutely necessary in the case of a level weir with a solid crest, as it is generally not advisable to attempt to impound much further storage above the top of such a weir. Should a sudden and intense flood come down after the temporary crest has been formed on the permanent one of the level weir, there will not usually be time to remove the upper crest. The flood-absorptive capacity of the reservoir will then be considerably diminished by the increased storage effected by that crest, the reservoir surface may rise much above the calculated high-flood level, and thus there will be a great risk of the failure of the whole work.

The case is different with a stepped waste-weir which has been properly designed with sufficient discharging power below its permanent crest level by means of under-sluices and automatic gates; moreover, the weir arcade will permit of the speedy removal of its temporary crest. With it the storage of the reservoir can safely be allowed to rise at the end of the monsoon to high-flood level, and thereby the maximum capacity of the tank can be secured without any additional increase to the height of the dam. In the event of a sudden and intense flood coming subsequently, it can be dealt with by opening the under-sluices and automatic gates as soon as it is seen that it is likely to occur, and, thereafter, by removing the temporary crest if

this becomes necessary. As the flood subsides, the temporary crest can be refixed and the gates and sluices closed so that the tail of the discharge may be impounded in order to restore the full storage capacity.

As an example of the saving effected by the use of the stepped waste-weir, the estimates in connection with the Máládevi tank project may be given. The original level waste-weir design was estimated at Rs. 51,430, but, on account of the deeper foundations subsequently found necessary, it would probably have cost at least Rs. 70,000. The stepped waste-weir design was estimated at Rs. 1,56,007, or, say, Rs. 86,000 more. The full-supply storages effected by the two forms are:—

	Storage in million cubic feet.
Level waste-weir crest, R.L. 188·00 .	3,385·602
Stepped waste-weir crest, R.L. 200·00 .	5,112·353

The latter, therefore, stores 1,726,751 million cubic feet more than does the former, and the capital value of this (at Rs. 238 per million cubic feet, the rate of the total storage of the project with the stepped waste-weir) is Rs. 4,10,967, thus giving a net saving over the design with the level weir, of, say, Rs. 3,25,000. The two projects compare exactly in respect of the dam, as for each the high-flood level was taken as R.L. 200·00.

Another way of comparing the two forms of weir would be to take them with the same full-supply storage at R.L. 188·00. In regard to the stepped waste-weir design there would, in consequence of its reduced height, be some saving in the cost of the masonry and some excess in that of the excavation of the tail channel, but, on the whole, there would not be much difference

in the cost of the weir itself. The saving in the cost of the dam by the reduction of its top level from R.L. 207·00 to R.L. 195·00 (7 feet above the combined high-flood and full-supply level) would, in this case, amount to about Rs. 2,74,000.

A later design for a level waste-weir 1,575 feet long was estimated to cost Rs. 1,83,020, or Rs. 27,000 more than the stepped waste-weir. Its full-supply level was lower by 2 feet, and its full-supply storage capacity less by 311·557 million cubic feet, than that of the stepped waste-weir design. This diminished amount of storage, capitalised at Rs. 238 per million cubic feet, is equivalent to a loss of Rs. 74,000. Moreover, as the dam had to be raised 7 feet higher to R.L. 214·00 with this design, its cost was increased by Rs. 1,95,000. Thus, this level waste-weir virtually costs Rs. 2,96,000 more than does the stepped waste-weir.

In the above comparisons has been taken into account only the storage rate due to the stepped waste-weir design; had the larger storage rates of the other design thus been taken instead, the saving due to the stepped waste-weir would have been increased considerably. By all these estimates the stepped waste-weir is seen to be much more economical than the level one: it secures this saving in addition to the other advantages mentioned in paragraph 189, p. 262.

Owing to the largely increased capacity of storage reservoirs at the higher contours above that at the lower ones, and to the rapid increase of the quantity of earthwork entailed by the raising of the dam, it seems probable that in every instance the adoption of the design of the stepped waste-weir will lead either to greatly increased storage or to a greatly diminished cost of the reservoir.

CHAPTER IV.

THE OUTLET.

I. General Remarks.

196. The Object of the Outlet.—The outlet is the work by means of which the water contained in the reservoir is passed safely through, over, or round, the dam, so that it may be utilised for the purposes for which the storage has been effected. In the smallest native tanks the outlet is simply a cut through the bank, which is opened when water is required for irrigation, and is closed by embankment when supply is no longer needed. The objections to such cuts are that the discharge cannot be controlled sufficiently during ordinary occasions, and that in the event of high floods or of neglect, breaches are apt to be formed by which the embankment is damaged and storage is lost. In a large work such a simple contrivance is quite out of the question, on account of the depth and of the pressure of the water, while, even for the smallest work, it is desirable to provide proper means of regulation and control.

197. The Location of the Outlet.—From one point of view the outlet ought to be placed at the head of the most suitable alignment for the distribution works. As, however, the cost of the first section of this alignment is relatively small compared with that of the dam, economy in connection with it is of much less importance than the attainment of the safety of the embankment, and the first consideration should therefore be paid to the proper location of the outlet with

respect to the dam itself when that is of earth. For a masonry work this matter is of less importance, as usually such a dam can have its outlet formed at the site best suited to the canal without endangering the main structure.

The best position for an outlet through a dam is at the centre of a saddle, or depression in the natural ground across the centre line of the dam, as then the embankment will settle symmetrically on both sides of it. There will thus not be any tendency to the formation of a crack or slip over it, and any leakage which may occur will naturally find its way at once to the centre of the depression, and will not lubricate the base of the dam beyond it.

The worst position for an outlet is on steep, side-long ground, and particularly on the side of the river gorge. As the outlet will there be placed unsymmetrically with respect to the longitudinal section of the dam, the earthwork on the lower side will have a tendency to move away from it, and any leakage at, or through, the outlet tunnel may lubricate the base of the embankment downstream and will then increase the tendency to movement of the superstructure above it.

When practicable an excellent position for an outlet is near one flank of the dam, or near a cross ridge from its centre line, as then it may be possible to construct the approach bank to the headwall (para. 214, p. 307) on natural ground, and not over the centre line of the outlet tunnel (Plate 13, Fig. 4). If placed over that centre line any failure of the dam through settlement may affect the approach bank.

In order to secure the best foundations, to cross the line of the puddle trench safely, and to prevent any settlement of the dam from affecting the outlet, it is

best to have the top of an outlet culvert some depth below ground surface.

These remarks apply to outlets the culverts, or channels, of which cross the dam, and not to tunnels or channels which lie wholly outside the dam (paras. 208 and 209, pp. 302, 303).

Wherever practicable, the outlet should be on the bank opposite to that on which the waste-weir is placed, so that the floods from the latter will not cross the channel from the former. This point should be taken into account when fixing the site for the waste-weir, as the location of the channel will generally be settled by the position of the land to be irrigated. Where, however, both works are on the same bank, the waste-weir floods will have to be passed over the channel in a superpassage, or, under it in an aqueduct, which will entail additional expense and may lead to difficulty.

198. The Number of Outlets.—Outlets may be sources of weakness in a dam, and it is therefore desirable to reduce their number as much as possible. In most schemes it will not be necessary to have more than one irrigation channel from the reservoir, but, where two or more have to be excavated, either on one or both banks of the impounded stream, the outlets should be as few as practicable. Where there are two channels on the same bank, the outlet for the upper one will usually be at such a high level that its construction will not be objectionable.

Where the channels are on opposite banks, it should be seen if one outlet could be made to serve both by a single main canal from the reservoir which would bifurcate into minor canals, one on each bank, some little distance below the dam and upstream of the outfall of the waste-weir tail channel. One of these

minors might have to be passed (in a large cross-drainage work capable of discharging the floods of the waste-weir) either above or below the tail channel: the other might have to be led across the valley in embankment with only a small work for local cross-drainage; or the two might have to be interchanged according to the location of the joint outlet. As far as the two minors are concerned, the cheaper arrangement would therefore be to cross the tail channel by the smaller branch canal: it would also be safer to have the joint outlet on the other side of the waste-weir. The saving by the reduction of one outlet would have to be compared with the extra cost of the cross-valley embankment alone, as the waste-weir tail channel crossing would, anyhow, have to be provided. It might be found economical to lower the sill of the joint outlet and to place that work nearer to the stream than would otherwise be thought necessary. Advantage should be taken of the work crossing the tail channel to make it serve also as a curtain across that tail to prevent retrogression of levels: this consideration might affect the selection of the site for that crossing.

199. The Level of the Outlet Sill.—The proper level at which the outlet sill should be placed will sometimes have to be regulated with respect to that of the land to be irrigated, but, as a general rule, this matter is not of great consequence, as the full extent of area to be brought under command can usually be secured by an extension of the irrigation channel. The more important points to remember are that:—

(*a*) Space should be provided below the outlet sill for the accumulation of silt in the reservoir (para. 42, p. 63);

(*b*) The capacity of the lower contours of a reservoir

being relatively small compared with that of the upper ones, it is generally not worth much extra expense to provide the means of tapping and utilising the lowest part of the storage;

(*c*) The lower the sill, the greater will be the cost and the insecurity of the outlet;

(*d*) The higher the sill, the quicker generally will the irrigation channel from it gain command, and the shorter and cheaper will be its course.

200. Subsidiary Uses of the Outlet.—The discharging capacity of an outlet is usually regulated, so that with a small head of, say, 1 or 2 feet, it may give the full supply required for the canal. It is, however, desirable that this discharging capacity should be increased considerably in order to secure the following advantages:—

(*a*) The rapid lowering of the water surface of the reservoir when this is required, either in the event of accident to the dam, or for the examination of the outlet sluices;

(*b*) The steadying of the rise of the water surface during the monsoon and the saving of the temporary or permanent waste-weirs by bringing into action, early and safely, the flood-absorptive property of the reservoir (para. 184, p. 253);

(*c*) The diminishing of the silting-up of the bed of the reservoir (para. 189 (*c*) and (*d*), p. 262);

(*d*) Assistance in the closure of the dam (para. 139, p. 191).

Taking as an example the Máládevi tank project, the outlet would have to discharge:—

	Cubic feet per second.
(1) To lower the reservoir 1 foot in a day below R.L. 177·00, the sill level of the under-sluices of the stepped weir (Plate 4, Fig. 2)	1,192

	Cubic feet per second
(2) To run-off $\frac{1}{100}$th inch per hour from the catchment	987
(3) To pass 15 inches run-off from the catchment in four months of the monsoon	514

For a catchment of moderate size, the increased discharge required to serve these purposes can be given by any of the ordinary forms of outlet, but for an extensive catchment the only form practicable, on account of the large volumes to be dealt with, is the headwall in the centre line of the dam (para. 205, p. 296, and Fig. 42, p. 310). An outlet of this type for the Máládevi tank project is illustrated in Plates 9 and 10; this has at high-flood level a discharging capacity of 7,027 cubic feet per second, which is equal to a run-off at the rate of 0·07 inch an hour from the catchment.

For a water-supply reservoir it will generally be necessary to have independent valves for these purposes, so that the increased discharge required may not interfere with the supply to the town (para. 228, p. 330, and Plates 13 and 14, Fig. 5).

II. Different Forms of Outlet.

201. Culvert under the Dam.—This form is illustrated in Plate 13 and Fig. 34 below. It is usually the cheapest type, and, when properly constructed, can be made perfectly safe.

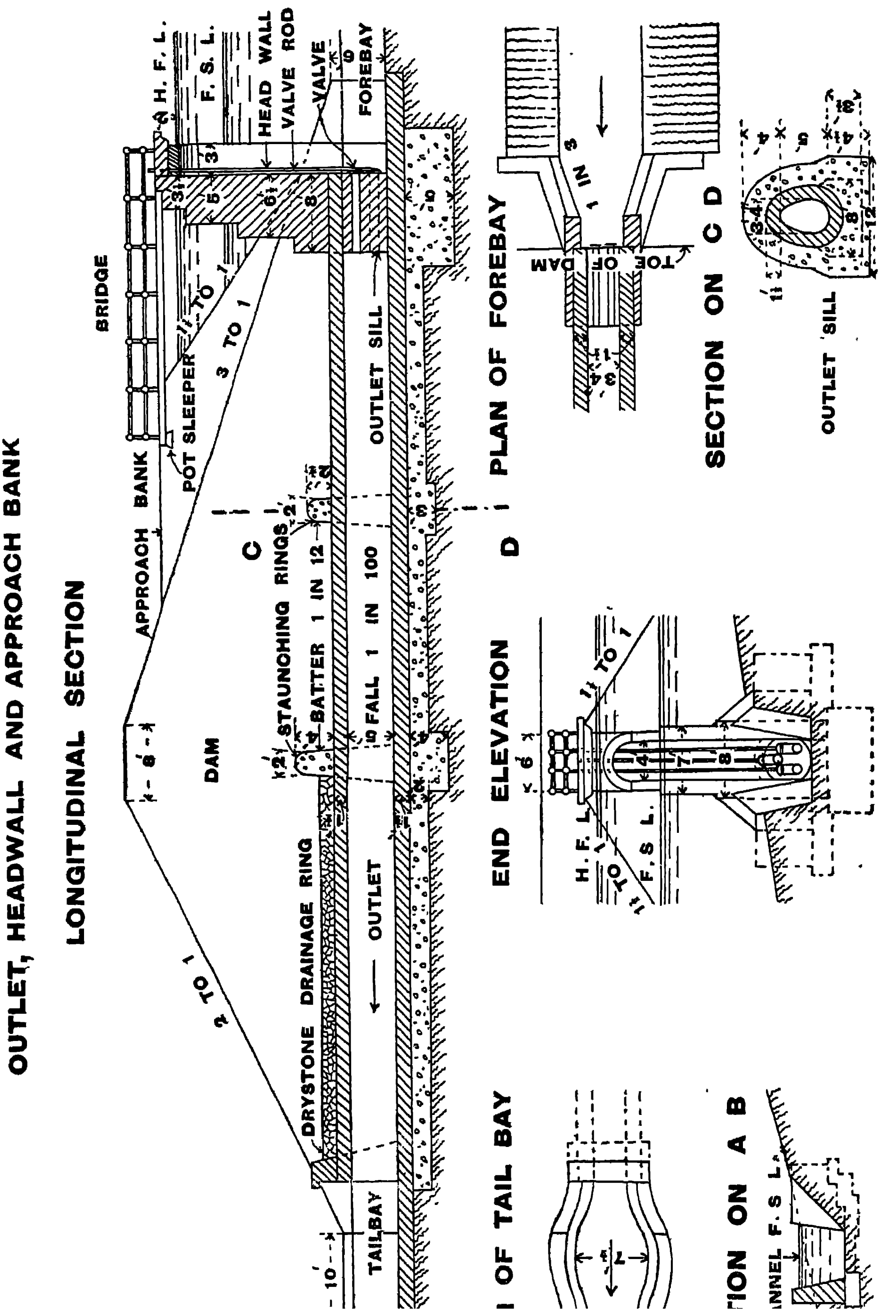
The following are objections which have been raised to it and the replies which may be made to them :—

(*a*) It forms a weak point in the dam. (This may be admitted in theory, but in practice this objection can fully be met by proper precautions in design and by good workmanship.)

(*b*) It cannot be inspected properly or repaired when necessary, as it is under a large mass of made earth. (The whole of the construction of the work being executed in the open, there should not be any difficulty in securing first-rate workmanship, and provided a durable material, such as stone, is used, there should not be any need of repairs, for the work being under the dam, is kept at an equable temperature and is not exposed to outside influences. A culvert of cast iron does not appear very desirable owing to the tendency of the material to rust, but, when one is constructed, it should be protected by an external ring of concrete and should be built up of rings in segments, each of which may be removed when decayed and replaced by a sound piece.)

(*c*) It is apt to be disturbed by the spreading out of the dam during settlement. (This is rather the fault of the embankment than of the culvert. No such spreading out should occur if the earthwork has been carefully executed, and none has happened in any of the large modern Indian dams. To prevent any possible failure of this kind, it is desirable to build the culvert in a trench and some depth below the surface of the ground, as the friction of the filling against the sides of the excavation will make the culvert practically independent of the dam. In such a trench the length of the culvert will also be reduced to a minimum. Its ends, being well buttressed by the tail- and fore-bays, should tend to prevent any motion outwards.)

(*d*) It is liable to be fractured where it crosses the puddle trench. (This is the most serious objection to this form of outlet. The puddle trench should never go under the culvert: the latter should have its foundations carried below the bed of the former, but,


OUTLET, HEADWALL AND APPROACH BANK
LONGITUDINAL SECTION
BRIDGE
H. F. L.
F. S. L.
HEAD WALL
VALVE ROD
VALVE
FOREBAY
POT SLEEPER
APPROACH BANK
DAM
3 TO 1
2 TO 1
STAUNCHING RINGS
BATTER 1 IN 12
FALL 1 IN 100
OUTLET SILL
OUTLET
DRYSTONE DRAINAGE RING
TAIL BAY
C
D
END ELEVATION
PLAN OF FOREBAY
1 IN 3
TOE OF DAM
SECTION ON C D
OUTLET SILL
I OF TAIL BAY
TION ON A B

where this cannot be done, a concrete trench should there be substituted for the puddle trench and on both sides should be well keyed into it. The culvert should have throughout its length a solid, unyielding, homogeneous foundation, and one not liable to be affected by the percolation of water. This foundation, when practicable, should be of sound rock, but, where this is not available, the culvert should be carried on a wide and deep concrete foundation. No reasonable expense should be spared to secure a thoroughly reliable work, and, if the natural conditions will not permit of this, the site should be rejected.)

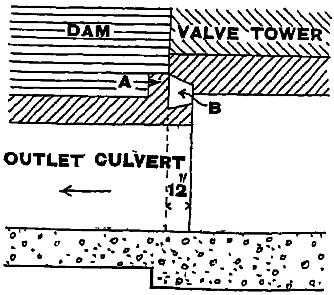
(*e*) A crack is likely to occur, owing to the unequal settlement of the different heights of the masonry, at the junction of the culvert with the headwall or tower. (Such a crack has not been formed at any modern Bombay reservoir, but, should it occur, it would involve only the leakage of water, which would pass harmlessly through the culvert, and would not constitute a danger to the stability of the dam. Fig. 35 below and Plate 14, Fig. 5, show how the formation of such a crack may be avoided: the upstream end of the outlet culvert is overlapped 12 inches by the concentric arch carrying the superstructure of the tower or headwall. At their junction, A B, the two arch rings are bevelled as shown, and, being disconnected from each other, can settle independently. When the masonry of the tower or headwall has settled finally, which will be before the reservoir fills, the annular wedge space can be made water-tight by running in cement grout.)

(*f*) Leakage is likely to occur between the culvert ring and the dam. (Particular care is necessary to avoid this. On the upstream side of the centre line

of the dam the junction can be made sound by placing a thick covering of good clay over the culvert ring, and by building this ring with minor projections and with staunching rings, as shown in Fig. 34, above. These staunching rings should completely surround the culvert and its foundation; they should batter in longitudinal and cross-section, and should have rounded tops, so that the earthwork may settle tightly on to them. The ring at the centre line of the dam should have a greater projection than the others, so that it may intercept any flow that has passed over them. The ring which is nearest the reservoir should be situated where it is not likely to be passed by a large amount of infiltration through the dam. Downstream of the centre line of the dam staunching rings are harmful as interfering with drainage; the culvert ring should there be cased with dry material, which will act as a drain and will lead away safely any leakage which has penetrated so far, and will not induce greater percolation from the reservoir. To prevent the flow through the casing from entering the dam, the former should be surrounded by watertight material. Also, the culvert should be built with cement or gauged mortar and cement pointed internally so as to make it quite water-tight. Where precaution (*c*) above is carried out, the culvert being constructed well below the dam, will not cause infiltration into that.)

FIG. 35

SECTION



(*g*) The valve tower, if placed in the reservoir and

connected with the dam by a bridge, is in an exposed position, and is liable to injury by ice or to become difficult of access in stormy weather. (In India there is no likelihood of ice affecting the works, and the stress of weather there is not sufficient to prevent attention being paid to the valves; these, moreover, can be protected from injury by being surrounded by cages or gratings.)

202. General Remarks on Outlet Culverts.

(*a*) *Sound Design and Construction necessary.*—This form of outlet is the one generally built; in Bombay failures have never occurred in connection with it, as the importance of securing absolutely good design and workmanship has always been realised there. Where the design is inferior or the construction bad, the following opinion doubtless holds good:—"Earthen [1] dams rarely fail from any fault in the artificial earthwork, and seldom from any defect in the natural soil; the latter may leak, but not so as to endanger the dam; in nine-tenths of the cases the dam is breached along the line of the water-outlet passages."

(*b*) *Water-tight Connection with the Trench.*—The trench excavated for the culvert should be considerably wider than the masonry, and should be taken out with slightly sloping sides to give space for a good thickness of puddle, so as to ensure that no leaks are formed along the culvert during the settlement of the earthwork. In some cases a concrete casing has been interposed between the ashlar arching and the puddle, or has been allowed entirely to supersede the latter. Every junction of dissimilar materials is, however, liable to produce a leakage plane; this may happen

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[1] *The Engineer*, Vol. lxiii., p. 189.

with concrete and any natural soil, and is still more likely to occur with the smooth surface of the concrete casing and the puddle round it. Puddle will fit much tighter between the rough back of the arching (the projections of which will tend to prevent the formation of leaks) and the sides of the excavation, and, as a greater thickness of it can be used at the same cost as a thinner casing of concrete, it is better to employ it alone for surrounding the culvert. The upstream half of the culvert should be surrounded up to 2 feet above its crown with good water-tight material so as to prevent leakage from it when it is running full from penetrating into the dam. Above that level the remainder of the puddle trench should be completed with the ordinary filling. Some engineers object to puddle as they consider it increases the stress on the culvert ring. If this action is feared, the ring should be thickened to resist it.

(*c*) *Form of the Culvert Arch Ring.*—Rankine [1] states that the proper form for the line of pressure is the elliptic linear arch, in which the ratio of the half-span to the rise shall not be less than the square root of the ratio of the horizontal to the vertical pressure of the earth. He adds that the entire ellipse may be used as the figure of the arch, or, if necessary, the bottom may consist of a circular segmental inverted arch having a depression of about one-eighth of the span.

Another form which is generally adopted is the ovoid (Fig. 34, p. 288, and Appx. 25 (V.), p. 480), such as is used for sewers, but for outlets the section is inverted so as to secure the widest base possible, and this, at the same time, gives the culvert the largest discharging power. For easiness of construction the invert may be

[1] "Civil Engineering," 11th edn., Art. 297 A, p. 434

replaced by a flat paved concrete foundation of a depth more than would just contain the true section; in this form the bed of the outlet should be constructed with a central drain to pass off leakage (Plate 13, Fig. 5, and Plate 14, Fig. 5).

(*d*) *The Construction of the Pavement of the Outlet.*—Where a flat bed is adopted, it should be finished off with a masonry pavement which should be carefully constructed of large stones breaking joint, the side stones should be well keyed under the arch ring, and all should be set in cement mortar, so as to prevent any displacement by a large rush of water. For the same reason the stones should be laid with their longer axes parallel to the flow. The culvert in all cases should have a small longitudinal fall, of say 1 in 100, to facilitate drainage and the flow of the discharge.

(*e*) *The Size of the Vent of the Culvert.*—The size of the culvert will primarily be regulated by the requirements of its discharging capacity, but a very small area of aperture will not permit of inspection, and too large a one will tend to make the work fail under the enormous weight of the dam. For these reasons the limiting internal dimensions of an ovoid section may be taken as 40 inches by 60 inches and 72 inches by 108 inches (Appx. 25 (V.), p. 480). It is not desirable to have two or more culverts side by side under the dam, as this will weaken it. If more discharging power is required than the single culvert can give, recourse should be had to the type of outlet—the headwall in the centre line of the dam—described in paragraph 205.

(*f*) *The Thickness of the Culvert Ring.*—This will depend upon the size of the culvert and the height of the dam over it. The material of the ring should be

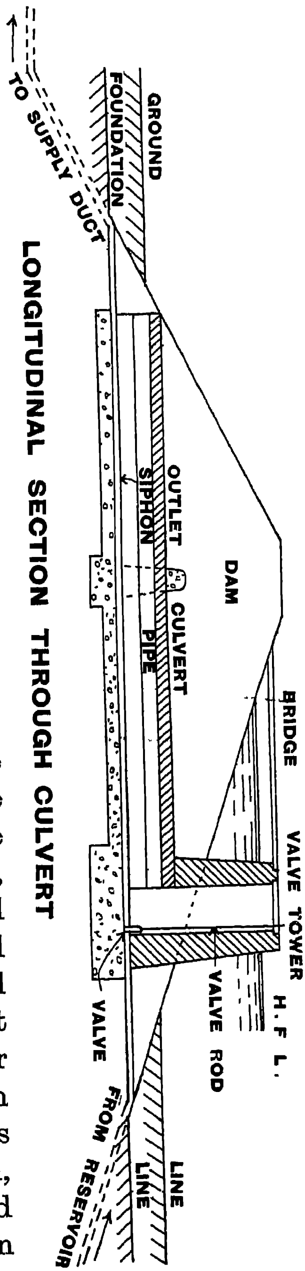
the soundest stone procurable, and for such stone the thickness of the voussoirs (Appx. 25 (V.)), may vary from 15 inches to 2 feet for dams under 50 feet high, and from 18 inches to 2 feet 3 inches for dams over 50 feet high. To strengthen the ring the concrete of the foundation should be carried up outside the masonry to the horizontal axis of the culvert.

(*g*) *Staunching Rings.*—These have been described in paragraph 201 (*f*), p. 289.

203. Pipe Outlet.—Formerly there were examples of pipe outlets in English dams of large size, but these led to failure, and this form of outlet, with the pipes embedded in the earthwork, is now never adopted for large works owing to the dangers attending its use. After the dam has been constructed, the outlet pipes cannot be inspected, they are liable either to break or draw under the weight of the dam, and water may leak along the pipes or through their joints. Pipes can, of course, be led through an outlet culvert for water supply schemes (Plate 13, Fig. 1) as in this position they are quite independent of the dams.

There are, however, numerous small native irrigation tanks where this form is the only one economically practicable. In such cases the pipes should invariably be laid on a hard, unyielding bed, should have staunching collars, and should be covered with the most water-tight clay procurable. For the smallest tanks earthenware pipes can be used; in some tanks in Madras and Ceylon concrete pipes have been employed; for the larger tanks, with a maximum depth of from 10 feet to 15 feet, cast-iron pipes will be required. These should be laid in a trench and should be packed all round with concrete, which will itself form the water passage if the iron pipe rusts away.

204. Siphon over the Dam. —This form of outlet, sketched in Fig. 36, has been built in England and America, but not in Bombay. It is best adapted to small discharges and is therefore not suited to the large ones required for irrigation, and still less to the maximum ones necessary to secure flood regulation. Moreover, it can be used only where the depth to be drawn off is small—probably 15 feet from the inlet end to the crest of the syphon is the practical limit, but to this can be added the depth below the reservoir surface at which that crest can be laid.



LONGITUDINAL SECTION THROUGH CULVERT

FIG. 36

The culvert in which the pipes are led under the embankment should have a wide concrete foundation, in order to secure uniform and small settlement, and should be laid in sound natural ground—*i.e.*, not in the heart of the dam. The necessity for having air-tight iron pipes in addition to the culvert, adds to the expense of construction, and will, in most cases, exceed the saving, consequent on

their use, of a shorter culvert and lower valve-tower. The design involves a loss of head to produce siphonage, and in many projects this would be a disadvantage.

205. Headwall in the Centre Line of the Dam.—This form of outlet is illustrated in Plate 9, and in Fig. 42, p. 310. It may be described as the insertion in an embankment of a short length of a masonry dam (the two being united by long staunching walls, one at each side); the passage for the discharge of the water through the work is kept open by means of four long wing walls. The headwall is pierced by outlet sluiceways controlled at their upstream face by valves worked by lifting rods and capstans; these openings can be made as numerous and of as large a size as if the whole dam were of masonry.

Great care is necessary to prevent the creep of water along the upstream wings and staunching walls, and the latter should therefore be made long and with projecting cross staunching walls as shown in the drawings. As the staunching walls are only of light section, they are comparatively not costly; they should be battered on all faces and rounded off at top and should have their tops ramped down away from the headwall, so that their length at different heights is in proportion to the pressure of water due to its depth which they have to withstand. They may be said to act as masonry core walls (para. 58, p. 82) uniting the headwall to the earthen embankment.

In Fig. 42 the wing walls are shown as purely masonry ones, and, when these are adopted, projections and recesses should be formed at the rear faces of the upstream ones to unite them with the embankment and thus to prevent the creep of water along them. On the downstream side of each of the staunching

forks should be a drain leading percolation water out of the dam (Plate 9, Fig. 3). In Plate 9 the wings are partly of masonry and partly of drystone for the sake of economy. The drystone should be constructed as described in paragraph 129, p. 179, and will thus be fairly water-tight and will unite with the earthen embankment.

The foundations of all the walls should be on rock, and should be as good as those required for a masonry dam. If the headwall exceeds 50 feet in length between the wings, its section should be that of a masonry dam of the same height, but, if the wings are closer to each other, they will act as counterforts, and the headwall may be made somewhat lighter.

This form of outlet should preferably be located in excavation in a saddle, as thereby the length of the wings can be reduced; this position will also be the best one for the location of the temporary waste-weir. If that work is excavated there, it will generally be better to build at it a central headwall than any other form of outlet.

206. The Advantages and Disadvantages of the Headwall in the Centre Line of the Dam.—The advantages of this form, permitting as it does of the use of large sluices, are :—

(*a*) The work, being wholly in the open, can easily be constructed, inspected and repaired.

(*b*) The reservoir can be kept low during the early part of the monsoon, when the floods are most silt-laden, and excessive silt deposit can thus be prevented.

(*c*) The reservoir can be lowered rapidly should an accident occur, or should the sluices require examination.

(*d*) As the full storage of irrigation reservoirs is not

required for some years after their construction (*i.e.*, until irrigation extends), this form of outlet will make it feasible to keep the reservoir surface low at first and will thus allow the dam to consolidate under the minimum amount of infiltration. When extra storage is required (as during a season of scarcity), it can easily be secured by closing the sluices towards the end of the monsoon.

(*e*) The outlet can be located at the site of the temporary waste-weir, and, in the case of a high dam, during its construction it can be raised so as to form a higher temporary waste-weir. This will save the necessity for the formation of two or more temporary flood-escapes situated in different places (para. 138, p. 190).

(*f*) Sufficient discharging power can be obtained, so as to save the waste-weir from being called into action in all but large floods.

The monetary value of (*b*), the diminution of silting, may be calculated thus :—The average annual yield from the Máládevi catchment has been estimated at 19,359 million cubic feet, or, say, 15,000 million cubic feet in excess of the full-supply storage of the tank. Assuming, (para. 38, p. 59), that the volume of the silt is $\frac{2}{3} \times \frac{1}{1000}$ of the volume of the water which contained it, and that the headwall discharge prevents the deposition of one-quarter of this, the amount of silt got rid of annually by it would be :—

$$15{,}000{,}000{,}000 \times \tfrac{2}{3} \times \tfrac{1}{1000} \times \tfrac{1}{4} = 2{,}500{,}000 \text{ c.ft.}$$

From Appendix 7, one million cubic feet should irrigate four acres of land, producing an aggregate net final irrigation revenue of, say, Rs. 20. Therefore the $2\frac{1}{2}$ million cubic feet of storage saved would be worth

annually, say, Rs. 50, and this capitalised at 25 years' purchase would be equal to Rs. 1,250, which may be taken as the financial value of the reduction of silting calculated on moderate assumptions. The result in regard to revenue is not great, but it is certainly of importance to retard the rate of silting up of a reservoir and the consequent annual decrease of irrigation under it.[1]

The principal disadvantage of the type is that it will generally be the most expensive form. From Appendix 1, p. 347A, it will be seen, however, that in No. 1, Mukti tank, with a maximum depth of available storage of 41 feet, the cost of the ordinary culvert outlet was Rs. 26,368, while in No. 14, Mhasvad tank, with a maximum available storage depth of 24 feet, but with a much greater discharging power, the cost of the headwall outlet was only Rs. 28,392.

The more elaborate design for the Máládevi tank project, illustrated in Plate 9, was estimated at Rs. 1,22,372, which exceeds by, say, Rs. 1,00,000, the estimated cost, Rs. 21,420, of a design for an outlet of the ordinary culvert form for another project for this reservoir. This latter must have been much under-estimated, as 5·4 per cent. (para. 28, p. 47) of Rs. 12,00,000 (Appx. 15, p. 376), the cost of the reservoir, amounts nearly to Rs. 65,000. The outlet originally proposed consisted of a masonry outlet tower 12 feet in diameter, with five 2-feet diameter sluices, a masonry culvert under the dam, and a light iron approach bridge of two spans of 37 feet each.

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[1] Rs. 1,250 is the capitalised saving on one year's silting; for each succeeding year the saving will gradually decrease as the storage of the reservoir lessens by silting.

However, the estimate of the design for the headwall on the centre line of the dam :—

	Rs.
Includes the cost of the temporary waste-weir channel	10,000
Saves the construction of a 200-feet length of embankment, which would cost about	37,000
Saves the construction of another temporary waste-weir channel and the increased amount of embankment over it, which would cost about . . .	23,000
Total reduction . .	Rs. 70,000

The remaining disadvantage of this type is that it may admit water to the heart of the dam, but it should be easy to prevent this by good design and careful construction, as explained in paragraph 205.

207. Outlet Tower in the Centre Line of the Dam.[1] —This type of outlet is known in America as the "dry well"; it is best adopted when a masonry core wall forms part of the construction of the dam (para. 58, p. 82). A culvert is constructed under the dam, and, where it crosses the core wall, a rectangular tower having one, two, or more divisions in plan, is built up as the dam is raised, and in it are placed the valves controlling the discharge (Fig. 37).

The advantages of this form are :—

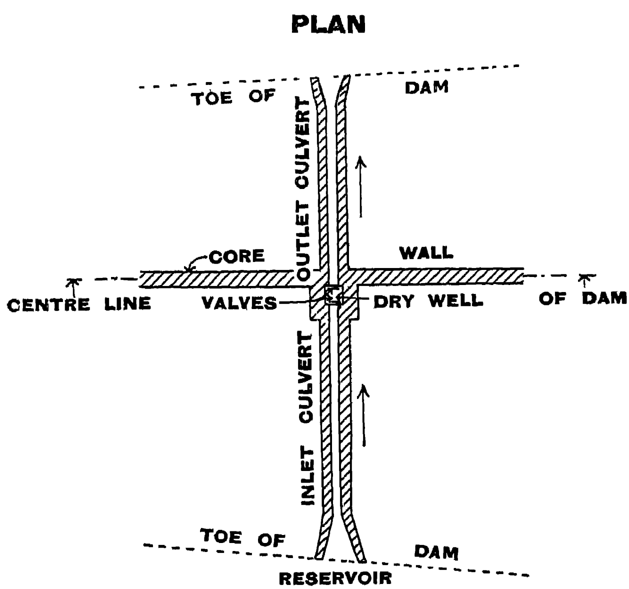
(*a*) It saves the cost of an approach bank and bridge ;
(*b*) It enables the valves to be easily approached in all weathers ;
(*c*) The tower cannot be injured by ice.

[1] "Minutes of Proceedings, Inst. C.E.," Vol cxxxii, p. 255.

The disadvantages attributed to it are :—

(*d*) Water is admitted into the heart of the dam. (In a work with a masonry core wall this is of small importance, as dependence is chiefly placed on that wall to cut off leakage) ;

FIG. 37



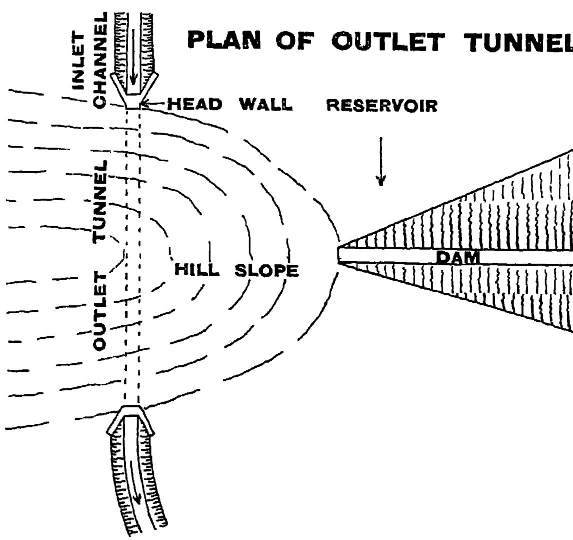
(*e*) The valve tower is liable to be affected by any unequal settlement of the earthwork, which may cause a leak to form and may force the lifting rods out of the vertical. (The tower will, however, be the strongest section of the masonry core wall.)

In an ordinary dam, without a core wall, these objections would be serious ones and sufficient to condemn the design for adoption.

208. Tunnel round the Dam.—This form of outlet (Fig. 38), being quite independent of the dam, cannot in any way affect that; it is, therefore, a very safe one and is being adopted to a considerable extent in English practice. The objections raised to it are :—

(*a*) Its great expense, which is generally not necessary, seeing that other forms of outlet can be made quite safe. In India, moreover, the scale of the

FIG. 38



natural features of the country is so large that in most cases the cost of the tunnel would be extremely heavy.

(*b*) The work, being underground, cannot be so carefully supervised as above-ground work. (This objection is not a very strong one, seeing that tunnels are well constructed in many situations, and that those for reservoirs will not usually be of considerable length nor of great depth below the ground surface.)

(*c*) The excavation of the tunnel, which should be in

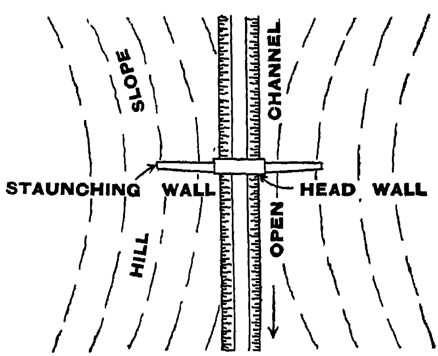
rock, is liable to cause fissures, which may lead to loss of water. (This, also, is not a very serious objection, as the fissures will usually not be of any great depth, and the water getting into them will, to a great extent, follow the line of the tunnel and emerge with it.)

(*d*) It is difficult to construct the tunnel headwall so as to make it have a water-tight connection with the excavation. (This objection can be met by providing the headwall with good staunching walls so as to unite it with the natural ground, and also by constructing a few staunching rings at intervals near the head of the tunnel.)

When the rock is sound it is not necessary to line the tunnel: if the rock is not sound, the expense of lining will usually be so great that it will be advisable not to adopt this form of outlet.

209. Headwall across an Open Channel outside the Dam.—This is a very simple form: a channel is dug round the flank of the dam, and across it is built a simple headwall in which are the regulating sluices (Fig. 39). The excavation of the channel (unless the spoil from it can be utilised for the construction of the dam) may be expensive, and thus this type is adapted only for works having a small full-supply depth above outlet sill. It shares all the advantages of the tunnel form, and, in addition, has that of

FIG. 39



being easily inspected during construction, and of being easily maintained thereafter. It was adopted in the case of the Medleri tank, No. 20 of Appendix 1, p. 347A; its cost there was Rs. 10,409, or nearly one-third that of the dam, which is very high.

210. General Remarks.—(*a*) *The Outlet Channel.*—The inlet channel upstream of the regulating sluices and the outlet channel downstream of the embankment should both be at right angles to the dam for a length at least equal to twice its height, as it is undesirable to have a deep excavation close to its toes. The former should thereafter be led directly towards deep water to shorten its length as much as possible, and the latter to ground sufficiently low to lessen the cost of excavation.

(*b*) *Fore- and Tail-Bays.*—On the upstream side the outlet channel should be confined by wing walls from the headwall (or outlet tower); these form the fore-bay (Fig. 34, p. 288). The longer these walls are the more expensive will they be, and the further will the water have access to the heart of the embankment; but the shorter will be the outlet culvert, the nearer will be the headwall to the dam, and the smaller will be the approach bank to it. As a general rule it is not advisable to have the wings very long; their tops should therefore not be advanced much further than where the dam is 5 feet above ground level, and their splay should be at 1 in 3.

Similarly, the tail-bay at the downstream end of the outlet should not have its end wall much higher than 5 feet above ground level. It may have its wings splayed at 1 in 3 or curved ogee in plan to form a discharge regulating basin at the head of the irrigation channel.

The wings of both bays should extend to where the prolongation of the dam slopes will meet the ground line 2 feet above the full-supply depth of the outlet, and should be returned into the natural ground (in a direction parallel to the axis of the dam), for a distance long enough to prevent any slips of the channel excavation from outflanking them.

(*c*) *Gratings.*—To prevent the entry into the outlet sluice of wood, rubbish, &c., which might interfere with its working, a grating should be fixed upstream of it (Plates 8 and 10). For large sluices this should be made of iron rails which can be utilised to form an inspection chamber by placing wooden planks upstream of them. For small sluices an ordinary grating should be fixed at a small angle to the vertical so that *debris* may not lodge in it. The waterway of all gratings should be considerably in excess of that of the sluices, so as not to obstruct their discharge.

(*d*) *The Outlet Gauge.*—It is convenient to have a gauge as close as possible to the tail-bay, so that the adjustment of the discharge let out of the reservoir may be effected as quickly as possible. If this discharge is passed down to the river to be picked up by the canal headworks, the gauge will conveniently be of the form of a clear overfall weir. If, however, there is no fall available for this purpose, or if the discharge is at once passed into the canal, the gauge will have to take the form of a gauging run. With either form it is advisable to have a table of discharges made out corresponding to the various depths of the channel, so that the actual quantity let out from the reservoir may at once be ascertained; the gauging run must, of course, be maintained to its correct original section, for which purpose that may preferably be formed in masonry or concrete.

(*e*) *Inspection of the Outlet Culvert.*—This inspection should be made twice yearly, once just before and once just after the monsoon, and all damage to the culvert, &c., should immediately be made good. As foul air may accumulate in the culvert, before the inspection takes place a full discharge should be passed down to clear out such air. Often the culvert can be inspected from the downstream end by reflecting sunlight along it internally by means of a mirror.

III. Regulating Works.

211. General Description.—The regulation of the discharge from the reservoir is effected by means of valves, or sluices, which control the inlet end of the outlet, and are themselves actuated by lifting rods, working up and down the face of the regulating headworks. The headworks will first be described; they may be classified as: (1) Outlet Towers; (2) Outlet Headwalls; and (3) Dam Slope Outlets.

212. Outlet Towers.—These are generally used for water-supply storages from which the supply has to be drawn at different levels in order that the clear supernatant water may be obtained continuously as the reservoir surface varies. If the water of the reservoir is excluded from the interior of the tower, the valves and lifting rods there can always be inspected. Should, however, the valves for securing double control (para. 215, p. 309) be placed on the outside of the tower, they cannot be examined, attended to, or repaired until the reservoir surface falls below them.

In plan, outlet towers are usually circular, and are flattened on the downstream side of the interior to form a seat for the guides of the lifting rods. A tower

square in plan is illustrated in Plate 13 ; it has a better architectural appearance than one circular in plan, and is in some ways more convenient than it, but will generally be more expensive.

The thickness of the walls of an outlet tower has to be designed with reference to the water pressure they have to withstand at different levels, so that leakage through them may be prevented. A much less thickness is sufficient to secure stability, the tower being subject on all external sides to equal water pressure, and a less thickness will also safely withstand the crushing effect of that pressure. For large towers it will be found economical to construct the walls with masonry facings and a hearting of very fine concrete (really a coarse mortar), well rammed in between them. To secure water-tightness, the work should be built, or at least pointed externally, with Portland cement.

Steps for inspection should be provided inside the tower from the top to the bottom, and these had better be arranged ladder-wise rather than spirally, as the latter system does not give a good hand-hold.

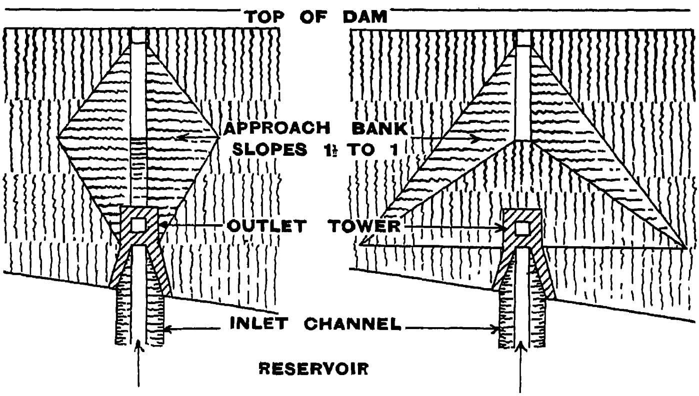
213. Outlet Headwalls.—For irrigation storages several outlet sluices at different levels are not necessary, as throughout the year the water is passed out from the reservoir at outlet-sill level, and for them a headwall is sufficient. The headwall consists of a simple wall on the reservoir face of which the lifting rods work (Fig. 34, p. 288). The capstans actuating these rods are carried at the top of the wall on a small platform, which is supported by arching springing from the top of the side pilasters of the wall, or by corbelling from the face of the wall.

214. The Approach Bank.—In order that the outlet headwall should not admit water near to the heart of the

dam, it is placed some distance on the reservoir side from the centre line of the embankment. To connect it with the main dam, an approach bank is thrown out from the latter, and from this a small foot-bridge is carried to the headwall (Fig. 34, p. 288). The top of the approach bank, the footway of the bridge, and the top of the headwall need not be raised more than 2 feet above high-flood level. The side-slopes of the approach bank, being of very short length, may be made at an

FIG. 40 FIG. 41

PLANS OF APPROACH BANKS



inclination of $1\frac{1}{2}$ to 1, as sketched in Fig. 40 and Fig. 41; the latter design, having the larger base, is the more secure form, and affords a better cover to the outlet culvert, thus diminishing infiltration into it. This bank should have its surface pitched all over except on the top. It should be made at the same time as the main dam, should be constructed with it, and should not be patched on to it subsequently. Owing to its pyramidal form it is not likely to slip, although its

side-slopes are steep. The end of the bridge resting on the approach bank is most conveniently supported by rails fixed in pot sleepers, which can be packed up to make good any settlement that may occur. Some long approach bridges are supported at their centres by timber or masonry piers, but the better design is to make them with bowstring girders in one span.

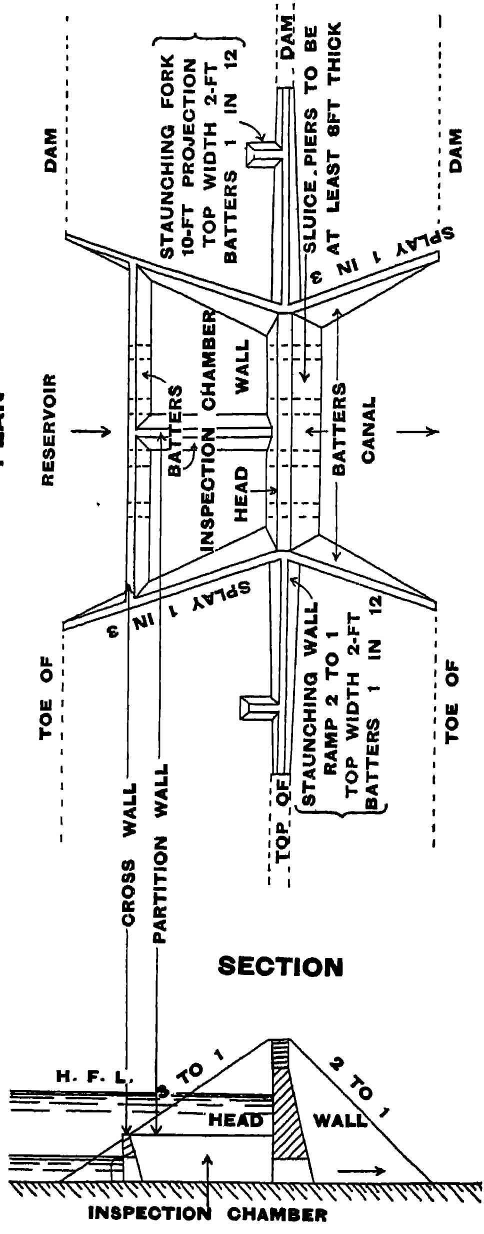
The advantage of having the approach bank leading to a subsidiary ridge and not to the main dam has been pointed out in paragraph 197, p. 282. Where this ridge offers good foundations, a masonry foot-bridge can be built from it to the outlet headwork with advantage to permanence and appearance (Plate 13, Figs. 2 and 4).

215. Double Control over Sluices.—If a single sluice is placed outside a headwall or on the reservoir side of an outlet tower, it cannot be inspected or repaired until the surface of the water falls below it. If, in the case of a tower, a second valve is placed upstream of the first one (which would then be inside the tower), double control is given to the outlet pipe to which they are fixed in common. By shutting off the supply to the downstream valve by means of the upstream one, the former can be examined and repaired, but the latter cannot be attended to until the water level falls below it. Thus, in ordinary designs, the outlet sluices of headwalls and the lowest external ones of outlet towers can be inspected only in the rare cases when the reservoir is emptied below outlet sill level; this extent of draw-off should, however, not be allowed to occur, especially in water-supply schemes.

The remedy for this defect is a simple one, and consists in making, just upstream of the headwall or tower, an inspection chamber by means of a cross-wall

FIG. 42

HEADWALL IN CENTRE LINE OF DAM



built between the wing walls of the fore-bay (Fig. 42 and Plate 13, Figs. 1 and 4). If necessary, and as shown in Fig. 42, a partition wall may be constructed in this chamber, so that one-half of that may be laid dry for examination, while the supply is allowed to pass through the other half. For large outlets with large sluices, the inspection chamber can be formed by placing wooden needles or planks in front of the grating which is primarily intended to prevent logs, &c., from obstructing the flow or from jamming the sluice (Plate 8, Fig. 3, and Plate 10, Fig. 3). For this reason this grating should be fixed in the piers some distance in front of the face of the headwall, and should be arranged with a slight vertical slope, so that *débris* may float up from, and not adhere to, it.

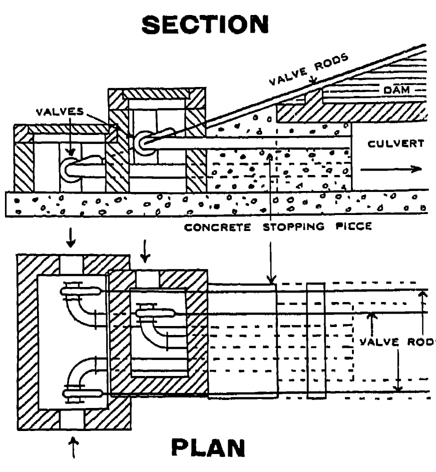
The top of the cross-wall, (or of the needles), should be raised to such a level that at it the storage contents of the reservoir are sufficient to last from the time of inspection (which would generally be a month before the commencement of the rains), until the first monsoon replenishment may be expected. At the bottom of the cross-wall would be a plain sluice opening, which could be closed either by a simple wooden shutter or by means of sand bags ; any leakage through it would pass down the outlet. By this arrangement the lowest valves and sluices could be examined, lubricated and painted, and, if necessary repaired, each year, while the uppermost ones would be attended to as the water surface fell below them.

216. Dam Slope Outlet.—In this form (Fig. 43) the expense of a masonry headwall is avoided, but it is applicable only to small reservoirs.

In this design the upstream ends of the outlet pipes are turned through a quadrant, so that the valves may

be circular, and not elliptical, as they would have to be were they placed at the mouths of straight pipes, and thus ordinary commercial patterns can be used. The valves are situated in small masonry chambers, and the pipes are led into the outlet culvert through a concrete stopping piece. The valve lifting rods pass through guides fixed to stones, supported on the slope of the dam, and are worked by gearing placed at its top.

FIG. 43



The objections raised to this class of outlet are :—

(*a*) The settlement of the dam may put the stone supports, and hence the guides, out of line with the proper direction of the lifting rods ;

(*b*) The valves cannot be inspected until the reservoir is dry ;

(*c*) Great force is required to work the long lifting rods (the length of each of which is that of the slope and not of the vertical height of the dam) ; these rods rest on numerous guides and, in the usual plan, actuate

elliptic-shaped valves of areas increased beyond those of circular ones. The guide friction can, however, be reduced by using rollers, and the valve area by the device above described.

IV. Valves and Sluices.

217. Ordinary Valves and Sluices.—For water-supply purposes, when the rate of supply is small, as it generally is, the commercial pattern of water-valve can economically be used. In this the valve travels up and down a screwed spindle fixed in a cast-iron casing, and therefore its position cannot be indicated by the lifting rod itself. For medium-sized irrigation works two or more of these may be used in conjunction with each other. For large irrigation works, where large discharges are required, special sluices are made. These consist of three parts—the fixed seating, the fixed guides between which the movable gate works, and the gate itself. Examples of these are illustrated in Plate 11 and Plate 12. (See also Appx. 18^{A}, No. 27, p. 431.)

All the surfaces in moving contact should be faced with planed brass, or gun-metal, to diminish friction, to prevent rusting, and to make a water-tight joint. As the seating for these large gates has to be made in more than one piece, great care must be taken to fix it to a perfectly true plane. The truth of the seating after it is fixed can be tested by stretching threads diagonally from its opposite corners and by then passing up horizontally another thread, which should throughout its course just touch the diagonal threads and the vertial sides of the seating.

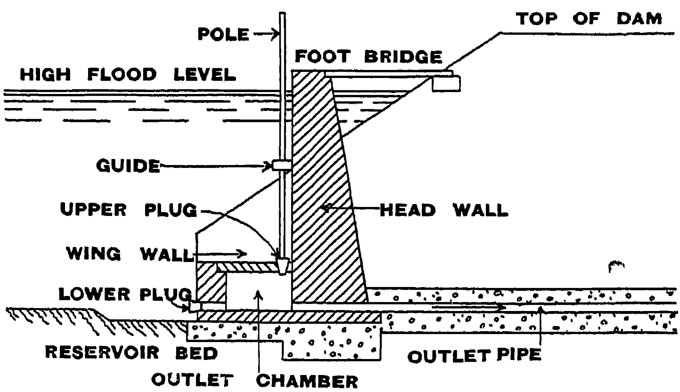
The masonry sill leading to the sluices should be altogether below the seating, and its upper surface

should be sloped downwards to the reservoir to direct the outflow upwards, so as to scour out any silt which otherwise might be deposited at the base of the seating. To prevent the sluice from jamming, it is best to make its bottom line part of a large circle and to bevel it slightly at the centre, as thus the gate will be guided on to the sill of the seating.

218. "Pole-and-Plug" Valves.—In some small native tanks the outlet head (Fig. 44) consists of a small

FIG. 44

PART LONGITUDINAL SECTION



masonry chamber roofed with a slab, which is pierced by a conical inlet hole the discharge of which is regulated by a conical wooden plug fitting into it and fixed to a pole. A second conical hole is made horizontally in the front wall of the chamber at outlet-sill level, which, in this case, is the silted-up bed of the reservoir, and this also is closed by a conical plug which is regulated by hand when the water level falls below the roof of the chamber.

This form of valve has been elaborated in iron as sketched in Fig. 45. It is simple, and can easily be worked and inspected; also, it enables the contents at the bottom of the reservoir to be drawn off with the maximum head. Under a great head of water, on account of its form, it would be difficult to work, and would be subject to much vibration.

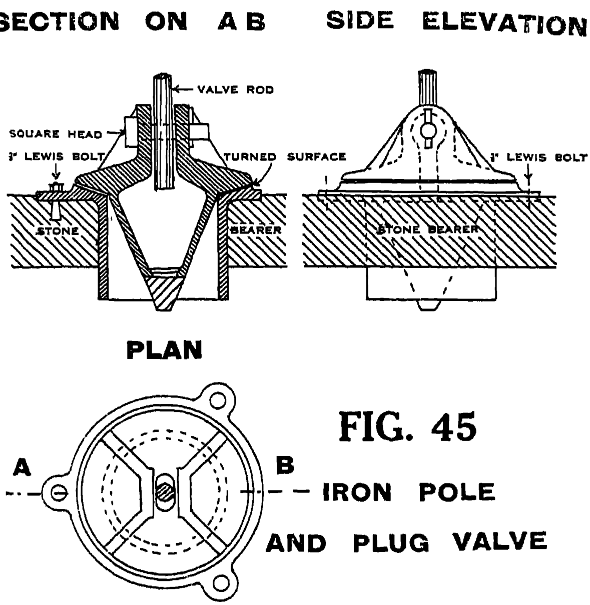


FIG. 45
IRON POLE AND PLUG VALVE

By using a number of these "pole-and-plug" outlets, at different levels in different chambers (Fig. 46), it is easy to maintain a practically constant discharge by lifting the different plugs as the water level falls. Similar regulation may be arranged for with simple plugs (Fig. 47) removable by hand.

Another form of "pole-and-plug" outlet on the principle of the "dam slope outlet" is sketched in Fig. 48. This can be used up to depths of 10 feet: it

FIG. 46

SECTION

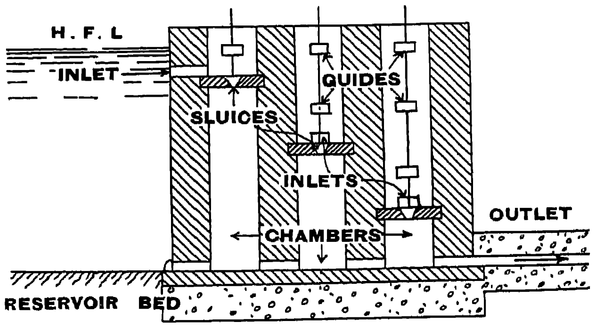
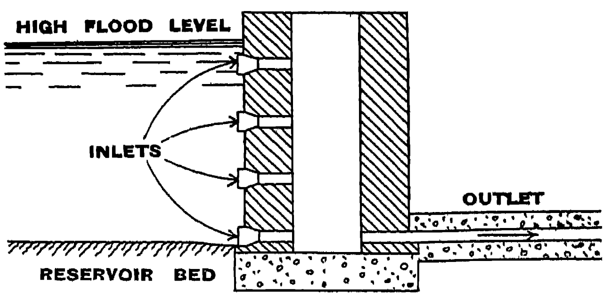


FIG. 47

SECTION



dispenses with the necessity for a high headwall, but, if thought desirable, its upper limb can be made vertical, a headwall can be built at it and the pole can be made to travel up and down this.

219. Equilibrium Valves—Working of Ordinary Valves.—The principle of the equilibrium valve is that as water acts on all sides of it, it is not kept on to its seat by water pressure, and therefore the force required

FIG. 48

DAM SLOPE OUTLET

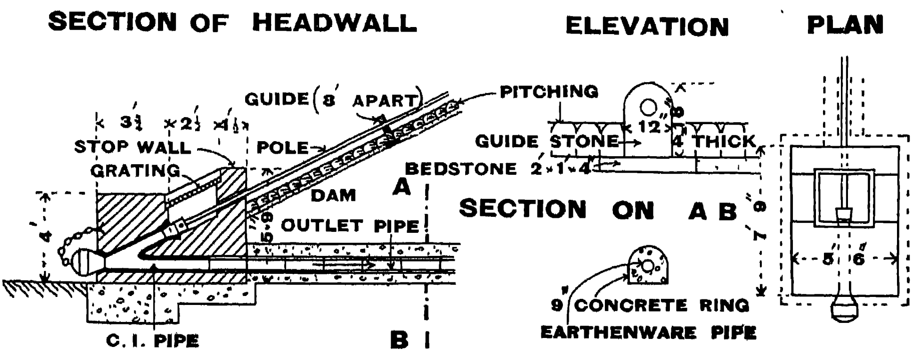
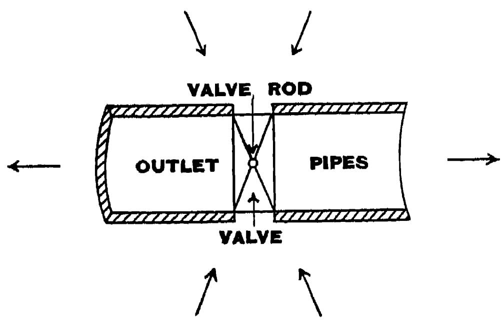


FIG. 49

SECTIONAL PLAN



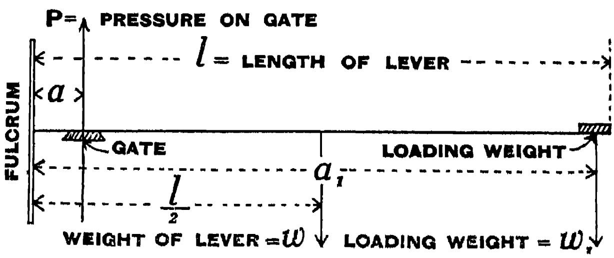
to work it is considerably reduced (Fig. 49). The valve is slightly tapered so that it fits water-tight on to its seat only when it is fully closed. Such valves are used in waterworks practice, where they frequently

have to be worked, but they are not adopted for irrigation schemes, as they are less water-tight, and require more space and therefore larger headworks than do ordinary valves. The expense of having a few extra men for the ordinary sliding gate during the few times it has to be worked for the regulation of irrigation is not great. A better arrangement for utilising the water pressure is to have a turbine by which the capstan heads can be revolved so as to actuate the lifting rods.

220. Testing Gates—Calculations for Gates.—Gates of any considerable size should be tested at the manufacturer's yard before delivery on to the works. For the test (Appx. 18A, No. 27 (xii), p. 435) the gate is laid horizontally so as to bear only on the longitudinal members of the frame, which will be its most unsupported position in practice (*i.e.*, when it is not fully closed), and weights are placed on it. A convenient arrangement is to put the weights on a series of levers (half right-handed and half left-handed), as this obviates the necessity for heavy foundations. Taking an actual case of a gate 5 feet broad and $7\frac{1}{2}$ feet high with its sill 29 feet below high-flood surface, the test, as shown in Fig. 50, was :—

FIG. 50

DIAGRAM OF FORCES



P = pressure on gate
w = weight of lever = 840 lbs.
w_1 = loading weight = 417 lbs.
l = length of lever = 25 feet
a = distance from centre of fulcrum to centre of gate = $3\frac{1}{2}$ feet
a_1 = distance from centre of fulcrum to centre of loading weight = $24\frac{1}{2}$ feet.

Then $P = \dfrac{wl}{2a} + \dfrac{w_1 a_1}{a}$

$$= \frac{\overset{\text{lbs}}{840} \times \overset{\text{ft.}}{25}}{7 \text{ ft.}} + \frac{\overset{\text{lbs}}{417} \times \overset{\text{ft}}{24\frac{1}{2}}}{3\frac{1}{2} \text{ ft.}}$$

$= 3{,}000$ lbs. $+ 2{,}919$ lbs.

$= 5{,}919$ lbs. $= 52 \cdot 85$ cwts.

As six similarly loaded levers were used, the total pressure on the gate was :—

$$6 \times \frac{52 \cdot 85}{20} \text{ tons} = 15 \cdot 85 \text{ tons on the centre line of,}$$

and $= 31 \cdot 71$ tons evenly distributed, over the gate.

Under the action of the test the gate should not be deflected appreciably. The amount of deflection would be ascertained by stretching a thread across the gate, and by measuring its distance from a fixed point when the gate was loaded and when it was unloaded.

The gate area $= 5$ feet $\times 7\frac{1}{2}$ feet $= 37 \cdot 5$ square feet.
The depth of the centre of gravity of the gate below the water surface $= 29 - \frac{1}{2}(7\frac{1}{2}) = 25 \cdot 25$ ft.

The water pressure on the gate $= \underset{\text{Sq. feet}}{37 \cdot 5} \times \underset{\text{Feet Head}}{25 \cdot 25} \times \underset{\text{Weight of water per cft.}}{62 \cdot 43}$
$= 59{,}114$ lbs. $= 26 \cdot 39$ tons.

The force required to lift the gate is :—

	Water pressure Tons		Coefficient of friction		Tons
The frictional resistance of the gate on the guides . .	= 26·39	×	0·3	=	7·917
The weight of the gate and the rod				=	1·528
Allowance for extras . . .				=	0·555
Total force required				=	10·000 tons

The compressive strain which can easily be put on the lifting rod by eight men is :—

$$\underset{\text{Men}}{8} \times \underset{\text{Lbs. Effort per man}}{20} \times \underset{\text{Circumference of capstan path}}{(2 \times 3{\cdot}14 \times 3{\cdot}5 \text{ feet})} \div \underset{\text{Pitch of screw.}}{\tfrac{3}{48} \text{ feet}} = \underset{\text{Lbs}}{56{,}269}$$

$$= \underset{\text{Tons.}}{25{\cdot}12}$$

This number of men should therefore suffice to lift the gate.

221. Suggested Arrangements for Gates.—The ordinary form of gate involves the construction of some kind of headwall, or tower, up the face of which the lifting rod works—in most cases under water, so that it cannot be inspected. If, instead of this, the valve were worked by bevel gearing placed at the downstream end of the outlet culvert, and the rod were passed through the culvert, the headwall would be saved, and the lifting rod could be inspected at any time. For such a design the valve could be given either a horizontal or a vertical motion ; the upstream end of the valve rod would be stepped into the valve seating, and would have a pinion engaging with rackwork fixed to the valve.

To reduce the force required to overcome the weight of the ordinary valve and rod, a counterpoise might be used, or if the gates are at different levels they might be coupled in pairs to balance each other, the upper one falling and the lower one rising to open the sluices, and *vice versâ*.

222. General Remarks.—(*a*) *The Outlet as a Flood-Regulator, &c.*—As explained in paragraph 200, p. 285, the outlet can be used as a flood-regulator, and also to lower the tank level rapidly. For these purposes the sluices should be made larger than is necessary only for the supply of irrigation water, or additional sluices should be provided.

(*b*) *The Outlet as a Source of Water Power.*—In paragraph 219, p. 318, it has been stated that the outlet sluices might be raised by means of a turbine worked by the pressure of the water in the reservoir. The pressure of the reservoir might also be utilised to furnish water power for industrial concerns, especially when the discharge is returned directly to the river and is not at once sent down the canal, as then there will be greater head utilisable. The water thus made use of will afterwards be available for irrigation. Unfortunately, owing to the isolated position of most reservoirs, manufacturing operations cannot usually be economically conducted near them, but with the advent of electrical installations this difficulty may be overcome. Another objection to making thus a further use of the stored water is that its discharge will have to be regulated solely by the requirements of irrigation, and may therefore be too irregular for being utilized for a manufacturing operation. This might be remedied by having an auxiliary steam plant for use as a standby on occasions when the water power was not available. Seeing that

the country must develop, and that scientific applications must improve, it is desirable, when constructing an outlet, to provide means for supplying water power, as this cannot be done subsequently without greatly increased expense and difficulty (see turbine pipe and valve, Plate 10).

223. Lifting Rods and their Adjuncts.—(*a*) *Lifting Rods.*—The lifting or valve rod should be attached to the gate below its centre of gravity (Plate 12). For the largest form of gate it should pass through a cored pillar, forming part of the casting of the gate, and should be secured by a screwed nut below it (Plate 11, Figs. 1, 2 and 10, and Appx. 18 , No. 27, p. 433). For smaller gates the end of the lifting rod may be forged out into two straps bolted to the gate, and extending diagonally nearly to its edges at the bottom.[1] The object of these arrangements is to prevent the gate from getting during its travel a sideways motion, which would tend to make it jam in its seat.

In one form of rod (Plate 11, Fig. 2) its top has a screw thread cut on it which engages with the female screw of the capstan head, through which it passes and by which it is actuated. The length of the rod which passes through the top (square) guide is made square in section, so as to be held by it, and is thus prevented from rotating with the capstan head. With this form of gear the whole lifting rod works up and down, and its top is always visible above the capstan head, so that the position of the sluice connected with it can readily be ascertained.

In another form (Plate 12, Figs. 1 and 2) the top of the lifting rod proper is bolted to a cast-iron pillar having

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[1] "Minutes of Proceedings, Inst. C.E.," Vol lxxvi., Plate 2, Fig. 10.

a female screw cut in it, into which the screw from the capstan head works, and, by revolving, causes the pillar and the lifting rod to move up and down. The pillar is square in section externally, and passes through a special square guide which prevents its rotation. The weight of the pillar adds to the weight to be lifted, and as the screwed lifting rod does not travel through the capstan head, its motion and the position of the gate cannot be seen directly. To determine that position it is necessary to bolt on to the top of the cored pillar a "tell-tale" rod, which will always project above the top of the headwall and will move up and down as the gate rises and falls.

(*b*) *Guides.*—To prevent the lifting rod from buckling, it is made to pass at intervals through guides or plummer blocks. The lower guides (Plate 12, Fig. 5) have circular holes, but, as explained above, the uppermost one (Plate 12, Fig. 7) has a square hole to prevent the rod from rotating. Care should be taken in spacing the guides that they will not interfere with the travel of the joints of the lifting rod, as these cannot pass through them.

(*c*) *Joints.*—The valve rods are liable to buckle, as an extreme amount of thrust comes upon them when the valves jam or do not work freely from any cause. It is therefore advisable to make them of mild steel and of somewhat fuller dimensions than are required merely for actuating the gates when they are working freely. In particular the joints are apt to give: for rods up to 2 inches in diameter the ends of the lengths should be forged out flat and formed into lap joints, secured by bolts passing through them: for rods of larger diameter their ends should be scarfed, and the joint should be confined in a strong collar and secured by slightly

tapered bolts passing accurately through the ends of the lengths and the collar (Plate 11, Figs. 16 and 17).

(*d*) *Stops.*—The positions of the gate when fully open or fully shut should be fixed by stops, and arrangements should be made to indicate visibly at the capstan when the gate has reached them so that the working of the capstan head may then be stopped.

224. Capstan Heads.—Plate 11, Fig. 4, illustrates a form of capstan head through which the head of the lifting rod rises and falls, and Plate 12, Fig. 1, one in which it only rotates horizontally. In each instance the capstan is bolted down to its foundation and its internal brass lining is free to rotate. In the former, the screwed head of the lifting rod works up and down through the internal brass, which is cut with a female screw. In the latter, the lifting screw has annular collars on it, which engage in recesses in the brass lining and prevent it from moving vertically. In this form, below the capstan head the lifting rod is cut into a screw which engages with a female screw in the head of the hollow pillar forming the top of the lifting rod proper, so that, as the lifting screw revolves, the hollow pillar is either drawn up or forced down.

In Plate 12, Figs. 8, 9 and 10, is shown a better design, by the late Mr. La Trobe Bateman, Past President Inst. C.E., in which the lifting screw works against a bed step and is separate from the lifting rod; the top of the rod is pinned to a long link carried by a movable stud engaging with the lifting screw. Any deviation of the lifting rod from the vertical is thus not communicated to the screw, and grinding action is thereby avoided.

CHAPTER V.

MISCELLANEOUS.

I. Water-Supply Schemes.

225. Proximity of Storage to the Town necessary.—Owing to the expense of conveying water in a sanitary duct from the storage reservoir to the population to be served, it is necessary in India, for all but very large towns, to select the site for that reservoir within a few miles of the town which it has to supply. The choice of site being thus restricted to a small area, the probability is that an economical one will not be available, and that the cost of storage will therefore be high. This is not a matter of so much importance in the case of a water-supply scheme as it is in that of an irrigation project, as for the former higher water rates can be charged. Moreover, although economy is desirable for a water-supply scheme, for it the quantity of storage required will be comparatively small, while the expense of distribution works will be great, and therefore the proportion which the cost of storage will bear to that of the whole project will generally be less than it will be in the case of irrigation reservoirs.

226. Amount of Storage, &c., required to ensure Certainty of Supply.—The amount of storage required for a water-supply scheme is much less than is necessary for irrigation. Taking the daily consumption per head as $12\frac{1}{2}$ gallons, and making an allowance of one-third of this for loss by evaporation (Appendix 7, p. 360), one million cubic feet of storage will suffice for one year for

the requirements of 1,000 persons, an amount which is calculated in that Appendix to be sufficient for the irrigation of not more than four acres. As, however, it is imperative that the water-supply for a town should never fail, it is necessary that for it there should be a storage sufficient to last for two years: for irrigation schemes this extent of storage is not wanted (end of para. 23, p. 40).

To secure this certainty, the catchment area should be ample, and, if the natural one is not sufficient, the run-off from subsidiary catchments (para. 9, p. 13) should be impounded in subsidiary reservoirs or diverted into the main one by feed channels. An additional advantage of a subsidiary storage reservoir is that if it is large enough, or if the main one can be arranged to feed it, as well as to be fed by it, either of them can, when necessary, be used exclusively in parts of alternate years. The one then not in use can be drained and its bed ploughed up and allowed to desiccate, which will improve its sanitary condition.

Irrigation should, as a rule, not be combined with water-supply schemes as the demands for town consumption, on account of their superior importance, must first be met. In years of deficient replenishment this may prejudice the working of irrigation as only the balance storage, which then will be little, can be allotted to it. It is only when the town supply is small relatively to the requirements of irrigation, and the total storage is designed as ample for both, that the combination can safely be made.

As for irrigation the draw-off is not constant, being regulated by the nature of the seasons and the requirements of the crops (which, for instance, do not need water after heavy rainfall), it may not be advisable to

combine a scheme for it with one for a commercial purpose (*e.g.*, lighting, power, &c.) for which a regular supply is wanted. When, however, the combination is made, it will be necessary for the latter to provide a "stand-by", either by a supplementary reservoir or by an engine, which would be brought into action when there was no demand, or a lessened one, for irrigation supplies.

In all ordinary situations the reservoir can be formed by means of an earthen dam, but, should there be exceptional circumstances rendering this in the least degree a risky form of construction, a masonry dam should be built instead, so as to gain perfect security of supply.

227. Sanitation of the Catchment Area.—Conservancy of Villages in the Catchment Area.—Filtration.— In respect to the amount of the yield of the catchment, there is no difference between the physical conditions which should obtain for a water-supply reservoir and for an irrigation one (para. 7, p. 10). In the former there is, however, the additional necessity for securing a clean, sanitary drainage area. The only absolutely safe gathering ground would be one consisting entirely of uninhabited, uncultivated, unpastured, and barren soil or rock of an insoluble nature. Such can practically never be found, and it is therefore necessary in a good water-supply scheme dependent upon a storage reservoir, to have recourse to filtration to remove the defects due to the nature of the drainage area. It is, however, most desirable to make the catchment as sanitarily perfect as possible. There is, of course, pollution from manured lands, but the amount of unburnt manure used in India is usually small, and the fact that it is ploughed into the ground will still

further reduce its potency for evil. The most harmful thing in a catchment is a collection of human inhabitants with their animals, and the pollution from these increases with their number and their nearness to the reservoir or to its tributary streams.

Probably the simplest effectual way of dealing with a village in a catchment area would be to surround it and the places reserved for defæcation by an embankment or by catchwater drains. The drainage from this area would then be led to a small tank with a storage capacity at least equal to one-fourth of the maximum annual yield of its catchment and this storage could be utilised for the irrigation of fields, also embanked round, and the tank emptied when necessary, say, four times during the monsoon. The embankments round the village and the area thus to be irrigated, and the dam of the small tank would all have puddle trenches under them, and would be constructed perfectly staunch. During the fair weather the bed of that tank and the lands irrigated from it would be ploughed up and allowed to desiccate. This irrigated area should be acquired, and might be let out on liberal annual leases, coupled with the condition that the lessees would be held responsible for the proper working of the system. Such a treatment would interfere little with the habits of the people, but would, of course, involve careful supervision.

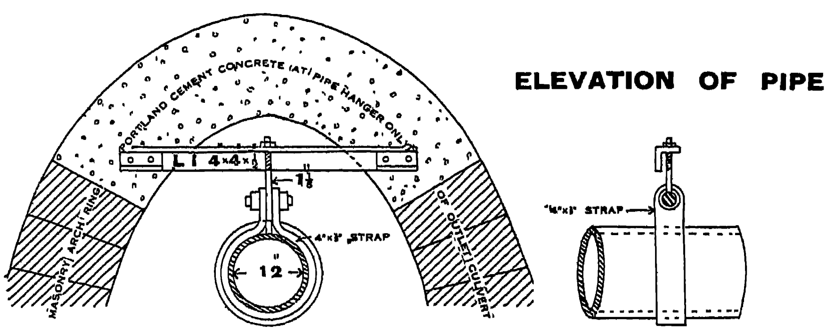
During the construction of the main and subsidiary storage reservoirs for the water-supply scheme, the workpeople should be camped below the dams, and should not be allowed to resort upstream from them for natural purposes.

Where filtration is adopted, it will generally be best to place the filters close to the storage reservoir,

(*i.e.*, at the head of the supply main), so that only filtered water may enter the pipe system and the spread of pathogenic germs in it may be prevented. Moreover, in this situation the ground levels will probably be more suitable for the location of the filters, and there will be available for them a larger area with cleaner surroundings than will be practicable near the town itself. In schemes having open ducts from the reservoirs, the filters must, perforce, be placed near the town, as these ducts are themselves liable to contamination.

FIG. 51

CROSS SECTION AT PIPE HANGER



228. The Works of a Water-Supply Storage Reservoir.—In respect of the dam and waste-weir, the design for the storage reservoir for a water-supply scheme will be the same as that for an irrigation project.

For the outlet it will be necessary to arrange to draw off the water from about 2 feet below the surface of the reservoir continuously as its level varies (para. 212, p. 306) and to have double control of the valves (para. 215, p. 309): for these reasons towers are generally

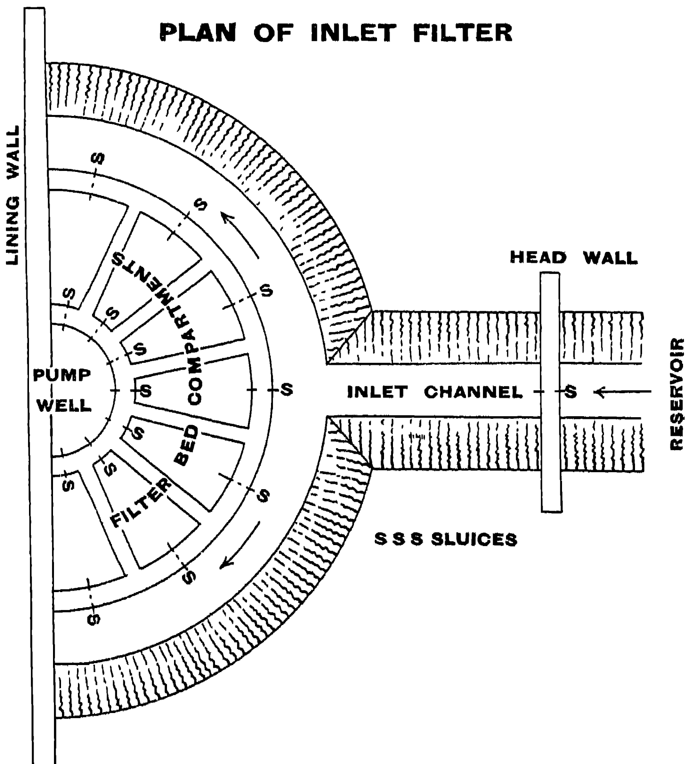
preferable to headwalls for water-supply outlets. In Plates 13 and 14 is illustrated an outlet for a water-supply storage reservoir. The water is drawn from the reservoir through a series of inlet pipes at different levels each of which is controlled by a valve in the interior of the water tower, and is protected externally by a fine grating in a bellmouth. The inlet pipes all communicate with a central stand-pipe, either directly or by means of short branch pipes at right angles to the inlet pipes, and these branches carry the guides for the lifting rods of the lower valves. The individual lengths of the pipes are designed to be of as few patterns as possible. Down-stream of the stand-pipe is a valve at the head of the outlet pipe, which thus gives double control to all the inlet pipes. Upstream of the tower is an inspection chamber, which provides treble control for the two lowermost valves. At the base of the tower are two large unwatering pipes, to be used when the tank level has to be rapidly lowered (para. 200, p. 286). All the valves are thus inside the tower, and can be inspected at any time and can be taken out and repaired when necessary, the water being then shut off from them by placing a tarpaulin, &c., temporarily over each inlet grating.

The outlet pipe (Plate 14, Fig. 5 and Fig. 51) is suspended from the crown of the culvert arch, so that its level can be adjusted by means of the suspending bolt. This is a convenient position for the main, as here it does not interfere with the drainage of the culvert nor obstruct the discharge of the unwatering valves, while the supply main itself can easily be inspected.

When the storage reservoir is below the level of the town, the supply for the latter will have to be pumped up from the former, and an outlet may thus become

unnecessary, which is an advantage (para. 198, p. 283). An arrangement for combining the inlet to the pumps with a filter is sketched in Fig. 52. The inlet channel is excavated till it reaches ground above the high-flood

FIG. 52



level of the reservoir, and across it is built the inlet headwall and the filter chamber each regulated by sluices and valves. The latter is designed with a series of independent compartments, each of which can be shut off from the others and its sand washed, &c.,

so that the operations may, if necessary, be made continuous. If the subsoil is suitable for the purpose, the water can be allowed to percolate through it to the filter, which will greatly aid in its purification. The water level in the compartments of the filter can be adjusted to suit that of the reservoir as it varies, and direct communication between the two works can be shut off by the valves of the sluices in the headwall separating them.

This design was made before modern investigations showed that the efficiency of a filter depends chiefly upon the bacterial scum which forms on the sand, and it might not be approved now on account of the small size of the compartments. However, these could be increased greatly beyond what is shown in the diagram without much cost, as only the head regulator wall and the upstream boundary wall of the filter would require to be made of heavy section to withstand, respectively, the pressure of the full head of water in the reservoir and the thrust of the ground. The filtration would be improved were a coagulant mixed with the inlet water.[1]

229. Utility of Water-Supply Storage Reservoirs as Famine Relief Works.—Owing to their situation near a large town, water-supply storage reservoirs form useful works for the employment of relief labourers, who can thus be easily supervised. The works, being comparatively small, can more nearly be completed during a single season of scarcity than larger ones,

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[1] Since this was written, quick-acting mechanical sand filters have been much developed. For working these, most of the bacteria are precipitated by coagulants before the water is admitted to the filters. The filters act principally by retaining the balance of the bacteria on their surface, and are washed frequently by filtered water passed upwards through them. This arrangement is efficient, quick-acting, clean, and compact.

and will be of greater public utility than an irrigation scheme which directly benefits a smaller number of the population. Even if the municipality concerned is subsequently unable to pay for the full value of the work done, the loss to Government may not be so great as if an unremunerative irrigation scheme had been undertaken instead.

II. Arrangement of Works.

230. General Arrangements.—Detailed notes of general arrangements are given in Appendix 22. Before commencing the actual execution of the work, great care should be taken to arrange everything so that due progress may be made uninterruptedly throughout the construction of the project. The workshop, stores, and office should be placed conveniently with respect to the work as a whole, and, for the sake of security and quiet, should be removed as jar as possible from existing habitations. The camps for labourers should be carefully selected with regard to sanitary conditions, a bazaar should be maintained under supervision, and all these should be placed under the charge of a medical officer. The camps should be set out in regular lines; each camp should be well separated from its neighbours, and should, as a rule, not contain more than 200 huts or 1,000 people. The sites should be properly located on high, well-drained ground, removed from undergrowth, and not too close to the working area. The huts should be erected on raised plinths 1 foot high, with main roads 10 yards wide and cross roads 5 yards wide. The huts may be of bamboo matting, grass, etc., as thus they can generally be constructed by the people with a little assistance in the shape of material. A hospital should be built near

the quarters of the medical officer, and so as not to be a danger to the camps, while, at the same time, not being far removed from them. The greatest precautions must continually be taken to conserve the drinking-water supply, and to keep the working area and camps as clean as possible, for which purpose conservancy and sanitary guards should be placed under the medical officer (Appx. 22, No. 29, p. 454).

231. Works Arrangements.—A large scale plan of the whole of the works and working area should be made, so that everything may be shown on it and may be arranged with proper reference to its importance in respect to the whole scheme. This should show the areas from which the soils required for construction can be obtained and the probable quantities available from them.

The most difficult and anxious part of the whole work is the closure of the dam, and all should give place to its construction. Works roads should be set out with reference to it; all materials should be stacked, and earth and muram required for its rapid completion should be reserved for it, close at hand.

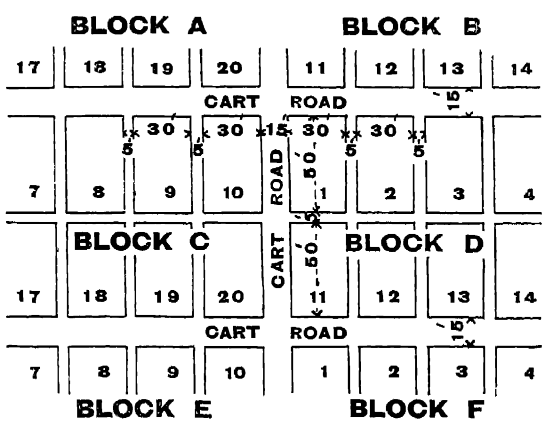
The area to be excavated for the construction of the dam should be set out methodically, so as to get the maximum amount of material from it at the cheapest rate, while, at the same time, ensuring a check on the measurements. For this purpose some such sort of arrangement as is sketched in Fig. 53 should be made. The whole area should first be divided into blocks separated by roads for carts, and each block should then be subdivided into pits to be excavated. At first these pits should be dug uniformly from 3 feet to 5 feet in depth, and should then be deepened gradually in steps, 1 or 2 feet deep, from the cart road, so that the

carts may enter each pit by moving down a ramp left at one side. An index plan should be kept with reference numbers for each block, which should itself have a reference letter, in order that the measurements may easily be traced at any time.

These pits are likely to be fouled by the workers, and a strict watch should be kept on them to prevent this from occurring. The most effectual preventive

FIG. 53

PLAN OF BORROW PITS



is, however, to arrange for proper latrine areas, at distances conveniently near to the working area, and to conserve them properly.

For all special works men should be specially selected and trained. The general arrangements should ensure that each evening every responsible man is fully acquainted with the work which he has to get done the following day; alterations of this programme should

be avoided as much as possible, as they cause confusion and delay, and thus occasion expense and retard progress.

III. Plant Required for the Works.

232. General Remarks.—In Bombay the earlier works were constructed with very little plant, reliance being placed upon labourers, pack animals, and carts for the conveyance of the earthwork on to the dam. The objections to this are that progress is not so rapid as might be, and that there is a greater liability to strikes and stoppage of work. The animals and carts no doubt help in the consolidation of the earthwork, but this can be done more effectually by rolling. General arrangements for "stores and tools" and "plant" are described under those headings in Appendix 22, Nos. 43–65, pp. 457–461.

233. Tram Plant.—For the rapid conveyance of earthwork it is advisable to have tramway plant consisting of light trucks, and rails with sidings, points, cross-overs, etc., and there should be sufficient of these stocked, so that some may always be available from store to replace damaged articles or to meet an emergency requiring quicker progress. The trucks should be light ones, chiefly of the side-tip pattern, with a few of the end-tip variety; the fittings should be as simple as possible, so that they may be renewed on the works when necessary and may not easily be injured. The trucks can be pushed by labourers or hauled by animals: it may in certain cases, such as the construction of the gorge embankment, be useful to haul them by means of stationary engines and wire cables.

In constructing the dam with tram plant, the rails should be laid on the embankment parallel to its axis, and should first be placed near one edge; the lines should then be shifted gradually over the top until they approach the other edge. As they are moved on, the earth deposited from the trucks should be mixed, levelled, rolled, and finally wetted, each operation taking place on strips parallel to the axis of the dam.

Rules for the management of tram plant are given in Appendix 22, No. 59, p. 460.

234. Rollers.—A light roller, weighing, say, a quarter of a ton per foot run, should first be used to consolidate and form on the deposited earth a surface upon which the ordinary roller, weighing, say, three-quarters of a ton per foot run, can work. Finally, in the case of all high and important dams, it is desirable to complete the consolidation by means of a 10-ton steam roller. Grooved rollers, split rollers, and light stone rollers for the top of the dam are noticed in paragraph 119, pp. 166, 167. Ordinary rollers should be of cast iron, not stone, and should be provided with scrapers to prevent them from lifting the earth.

235. Pumps.—Water should be laid on all over the works both for constructional purposes and for the supply of the labourers. As the lifts will be high for large dams, it is advisable to have for them steam pumps, piping, and 4-feet cube wrought-iron tanks connected in series as reservoirs for these purposes. Hand pumps are useful for lower lifts and for unwatering foundations; for the latter there should be a reserve of pumping power to deal with any unusual amount of water which may be met with, as in such cases everything depends upon the rate at which it can be got rid of.

236. Stone-Metal Crushers and Concrete Mixers.—These will be required only when there is a large amount of concrete to be made for the waste-weir and outlet; as a general rule hand labour will suffice for the preparation of material for these works.

237. Tools and Instruments (Appx. 22, Part 4, p. 457).—It is true economy to have good tools, especially on a large work, where it is certain they will be worn out rather than will rust away. There should always be a large balance of good tools in store to meet sudden demands, and to enable useless tools to be replaced without delay.

Similarly, there should be a reserve of surveying instruments, as these are peculiarly liable to injury on a large work.

IV. Maintenance of Works.

238. The Dam.—During the construction of the dam, careful observations by level and theodolite should be made, as described in paragraph 125, p. 173, at all high parts of the dam, and at such others where from any cause there may be any tendency to failure. These observations should be continued for at least a year after all movement has apparently ceased. During their continuance and after their cessation an annual record should be kept of the levels of the top of the dam taken on the chainage stones fixed on it. That top should always be maintained to its designed height, and any settlement that occurs should be made up before the succeeding monsoon.

The next most important matter to which attention should be paid is the drainage of the dam. Rain water should not be allowed to concentrate when flowing down the slopes nor to lodge anywhere near

the base of the dam, and all drains there may be should be kept running free from all obstructions. Wherever possible, a continuous register of the discharges of such drains should be preserved as a permanent record. The best test of the sufficiency of the drainage is that the ground downstream of the dam is dry.

The dam should always be kept clear of long grass and shrubs, the latter being carefully rooted out. The slopes should be maintained to an even surface so as to shed the rainfall regularly. Trees should not be allowed to grow on the dam nor within 30 feet of its toes. Some weeks before the monsoon commences, the slopes should have coarse vegetation on them burned; this will make the young grass spring up the better afterwards, and will also expose any rat-burrows, etc., and give time for their being cut out and refilled. Burning has, however, the disadvantage of destroying the finer grasses and thus of encouraging the growth of the coarser and less desirable ones. During the monsoon a permanent gang of labourers should be employed in preventing the guttering of the slopes and in filling up cracks, settlements, and rain scores. Cracks can best be filled by ramming into them a gritty and clayey mixture by means of chisel-pointed poles. The gang should be maintained at full strength for the first two monsoons, and thereafter it can gradually be reduced as experience dictates.

The pitching should be examined as the water level falls, and all loose stones, settlements, and other defects made good in a continuous system of repair. All shrubs should be rooted out.

239. The Waste-Weir.—All scour channels in the

tail channel near the weir should be prevented by curtain walls, boulders, etc., from cutting back towards it, and the approach channel should be maintained clear of obstructions to flow.

If there is a temporary crest, this should be removed as soon as the reservoir falls below permanent crest level, and all woodwork in connection with it should be dried, tarred, and stored. It is generally not advisable to attempt further storage from occasional storms later on in the year.

As soon as the sluices are laid dry, they should be oiled and painted, and, some time before the monsoon commences, the lifting gear should be tested, so as to see that all is in perfect working order.

The masonry should have all repairs effected as early as possible after the monsoon, so that the mortar may have the advantage of setting properly in the cold weather. All plants growing in the masonry should be rooted out as soon as possible. These remarks apply equally to the masonry works of the dam and the outlet.

240. The Outlet.—The culvert should be examined twice a year, some little time before the monsoon and just after its close, and all structural repairs should then be made good. The valves, their seats, and their rods, should be examined a month before the monsoon, and all parts in contact greased and others painted. The capstan heads and screws should be constantly oiled and protected from the weather; in the case of the screws this may be done by wrapping them round with oiled coir string. The ironwork of the approach bridge and headwall should be kept well painted, and the woodwork tarred, and any settlement which may take place at the approach bank should at once be made up.

241. The Reservoir.—The land boundary marks should be inspected and maintained. The plantations should be carefully attended to, and preparations for planting more trees made throughout the fair weather by the small number of guards employed to watch the plantations.

If silt clearance by ploughing is attempted (para. 36, p. 57), the ploughing should proceed continuously as the reservoir level falls and the ground dries sufficiently to permit of it. Field owners should be encouraged to embank their fields in order to catch silt; where there is not hard ground for the formation of small waste-weir escapes, pipes with proper inlets might be fixed at the flanks to lead the impounded water safely away.

The silt experiment lines (para. 39, p. 60), should be examined, and at intervals, say of five years, should be levelled over just before the replenishment of the reservoir is likely to commence.

Observations of rain gauges in the catchment should be made daily and recorded; daily records of the reservoir level and outlet discharge should be kept; flood observations should be taken, and observations to test the loss by evaporation and absorption should be carried out.

V. The Report, Plans, Estimates, and Specifications.

242. The Report.—(*a*) *General Description.*—The Report forwarding the plans and estimates for sanction should be as concise as possible, and repetitions should be avoided. The writer should, however, place himself in the position of the persons who will have to

study the Report without the advantage of detailed local knowledge, and should make everything as clear to them as it is to himself from possessing that knowledge. All statistical information should be given in a table, as in Appendix 26, p. 481, where it can be found in a moment, whereas, if it is buried in a mass of verbiage, it is not so easily traceable. It is not, necessary that the Report should discuss matters of common professional knowledge, as its main objects are to explain the reasons for selecting the site dealt with and rejecting others, and to describe the particular features of the scheme. The reasons for the rejection of these other sites should be given carefully so as to show that the whole neighbourhood has been investigated thoroughly, and thus to avoid a reference on this point by those who have to examine the scheme (para. 43A, p. 64).

(*b*) *Preparation of the Project.*—The extent of survey done is best explained by recording the result of the field work on the plans. The names of all those who have been connected with the drawing up of the scheme should be reported, so that they may be given all the credit which is their due, and so that proper weight may be attached to their opinions.

(*c*) *The Site and the Works.*—All peculiarities of the site; the nature of the foundations; the reasons for locating the different works at the places settled for them, and for choosing the types of works adopted in the project should be described; and the locality, quantity available, and cost of the materials required for construction should be noted.

(*d*) *Statistics of Rainfall and Yield.*—In respect to rainfall statistics, it is best to tabulate the daily fall during the monsoon months of as many years as

possible, rather than the total monthly or annual falls. From such a table may be prepared one showing falls each over 1 inch, which alone are likely to produce a fair amount of run-off. The statistics on which the yield of the catchment and the storage required have been calculated should be given in detail. It should be noted if rainfall records or river-gauge statements of discharge have been depended upon—the latter, it is scarcely necessary to say, are the better of the two, as they show the actual results of all the factors producing run-off.

(*e*) *Waste-Weir Floods.*—The method of the disposal of the waste-weir floods should be described, and it should be made perfectly clear that their course will be harmless to neighbouring lands and property, or, if they are likely to cause damage, that full provision for compensation has been made in the estimates.

(*f*) *Revenue Matters.*—The principal crops grown in the neighbourhood; the probable effect on cultivation of the introduction of irrigation; the nature of the soil of the land under command; the amount of manure and fuel available; the character and financial status of the cultivators; the peculiarities of the seasons and climate; the prospects of trade owing to the nearness of good markets; and the facilities afforded by communications by road and rail should be stated. The written opinions of the Revenue officers on all these matters should be obtained before the project is worked up, and should be attached to the Report.

(*g*) *Financial Return.*—When calculating the financial return, the results of an average year should be taken into account, that is to say, the storage, "duty," and area irrigated which are considered should be those of an ordinary year. The protective value of the work

during a drought year should also be estimated. Statements should be attached to the Report to show the probable annual expenditure on the construction and maintenance of the work and the financial results (gross and net) anticipated from it during the first twenty years after its completion.

(*h*) *Arrangement of Report.*—The Report should be divided into sections and paragraphs, and these should be carefully indexed for ready reference. At the end should be a series of statements giving the principal dimensions, costs, etc., of the project (Appx. 26, p. 481); the reservoir contents at each contour (Appx. 17, p. 382); rainfall and river gauge statistics; and waste-weir flood calculations (Appces. 12, 13, and 14, pp. 370–375).

243. The Plans.—Care should be taken to send up a complete set of plans, so that the scope of the project may easily be ascertained from them. It is a waste of time to make any but type drawings for minor works, as these may have to be altered during construction, and their detailed design can safely be entrusted to the discretion of the officers who will have to carry them out. The scales of the drawings should be chosen so that they may be as small as is consistent with showing the proper amount of detail in them; large, unwieldly drawings are difficult to deal with, and are usually unnecessary, as intricate parts can easily be shown by enlarged detailed drawings. The index plan should show all places mentioned in the Report; all roads, railways, irrigation works, and natural features; and the proposed canal system and the land irrigable by it.

The following is a list of drawings for the reservoir, which comprises all that will generally be required:—

1. Index Plan of Project.—Foolscap size (for insertion in the Report).

2. General Plan of Catchment.—Scale, 1 mile to 1 inch.

3. Contoured Plan of Reservoir.—Scale, 660 feet to 1 inch (to show all works and the waste-weir out-fall).

4. Land Plan (to show all land to be acquired).—Scale, 660 feet to 1 inch (this should correspond with the village maps).

5. Dam.—Plan, longitudinal and cross-sections; details of foundations and closure arrangements.

6. Waste-Weir.—Plan, longitudinal and cross-sections; details of automatic gates, sluices, and temporary weir crest.

7. Outlet.—Plan, longitudinal and cross-sections; details of tail- and fore-bays, approach bridge, valves, lifting rods, and capstans.

On every drawing, where required, all the foundations and the trial pits by which they were determined should be shown; the direction of the flow of water should be indicated by arrows; and all reduced levels, water levels, and dimensions should be carefully given, so that they may at once be seen on inspection.

Each plan should bear the name of the project and its own distinctive name; the year of its preparation; the estimated cost of the work; the names of the surveyor, designer, and draughtsman; and, where necessary, references to benchmarks and to the pages of the book in which the survey is recorded. It will make reference easy if the plans (tracings or blue prints) are folded foolscap size and bound in a series of pamphlets (or placed in small portfolios), each pamphlet, or portfolio, having outside an index to the drawings it contains.

After a work has been constructed, a set of completion drawings should be prepared; these should record exactly how it has been executed, major deviations from the original sanction should be clearly shown, and all foundation lines and levels should be entered.

244. The Estimates.—The detailed estimates should be prefaced by a Recapitulation showing the total cost of each work arranged under main heads (Appx. 15, p. 376).

The estimates of the works should then follow in the order of the recapitulation, and each should consist of a general description, the detailed measurements and an abstract of cost. The general description should be confined to explaining the drawings and to giving any tabular information there may be respecting them: the reasons for selecting the type of work chosen and other general particulars should appear in the Report itself. The measurements under the different subheads should be arranged throughout in the same order, so that the total quantities and costs of any particular part of the work may be ascertained at any time. The abstract should be made out so as to exhibit clearly the total estimated cost of each subwork, and the "contingencies" allowed for each should be added in each case to enable this to be done (Appx. 16, p. 377).

245. The Specifications.—It is desirable to draw up for each district a complete set of specifications which can be printed and attached to each project, thus saving trouble and securing as much uniformity as is desirable. The specifications should clearly distinguish between what is definitely settled and what is left to the discretion of the principal responsible

officer, and as much latitude as possible should be given to him to meet unforeseen requirements. Specimen specifications for earthwork and pitching are given in Appendix 18, p. 387, and for the waste-weir and outlet in Appendix 18A, p. 407.

COMPARATIVE STATEMENT OF DIMENSIONS AND COST OF STORAGE WORKS WITH EARTHEN DAMS IN THE BOMBAY PRESIDENCY (DECCÁN) REVISED UP TO 1894

(*Vide* CHAPTER I, PARAGRAPH 28, PAGE 47)

No	Name of Work	Data: Area of Catchment, Square Miles	Data: Average annual rainfall and estimated proportion of run-off to rainfall	Data: Fall of River above Dam, Feet per mile	Dam: Length of top of Dam, Feet	Dam: Maximum height of Dam, Feet	Dam: Depth from full supply to level of sill of Outlet, Feet	Dam: Mean sea level of sill of Outlet, Feet	Dam: Width of Dam at top, Feet	Waste Weir: Length of Waste-Weir, Feet	Waste Weir: Height of Calculated Maximum Flood over Weir, Feet	Waste Weir: Height of Dam over Weir Crest, Feet	Waste Weir: Estimated run off per hour, Inches	Waste Weir: Estimated discharging power of Waste Weir, Cubic feet per second	Description of Waste Weir	Tank: Area of Contours in millions of square feet: At full-supply level	Tank: Area of Contours in millions of square feet: At outlet level	Tank: Total storage capacity, Mill cft	Tank: Total storage capacity over sluice sills, Mill cft	Tank: Depth allowed for evaporation and loss due to all causes, Feet	Cost of Works: Dam, Rs	Cost of Works: Outlet, Rs	Cost of Works: Waste Weir, Rs	Cost of Works: Land Compensation, Rs	Cost of Works: Total, Rs	Cost (works charges only) of water stored per million cubic feet Col 26 — col 19, Rs	No	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
1	Mukti Tank, Khandesh	34 22	20 92 / 4	21 30	3,000	65 00	41 00	885 50	10 00	1,590	5 40 and 2 40	13 00	1 00	32,265	Two masonry walls	22 177	Nil	342 429	342 300	7 00	1,96,745	26,368	42,491	1,775	2,67,379	781	1	Storage for Lower Pánjhra River Works
2	Mherun Tank, ,,	3 70	22 20 / 4	45 28	1 848	41 20	34 35	[2] 107 00	8 00	200	3 00	9 00	1 00	2,226	Excavated channel	6 575	0 050	68 044	67 894	7 00	41,371	2,515	17,034	3,603	64,523	948	2	Town water-supply
3	Mhasva Tank, ,,	13 40	22 51 / 3	21 39	1,494	44 14	22 00	[2] 129 00	10 00	370	3 50	10 00	1 00	8 621	Masonry wall	18 348	0 790	160 962	158 571	3 00	56,682	5,147	4,061	4,039	69,929	434	3	
4	Hartala Tank, ,,	6 80	19 67 / 3	30 00	1,200	51 50	13 70	[2] 80 30	10 00	136	4 26	10 00	1 00	4,265	Excavated channel	2 057	4 592	171 609	134 868	4 00	27,638	508	2,541	45	30,732	179	4	Old work restored
5	Parsul Tank, Nasik	17 33	28 00 / 4	42 70	2,770	62 27	35 00	1,841 38	6 to 8	500	4 00	10 00	1 00	12,800	Masonry wall	6 620	1 011	124 500	118 700	4 00	1,31,226	7,360	10,827	1,656	1,51,069	1,213	5	
6	Sirsuphal Tank, Poona	23 00	20 48 / 4	20 00	2 188	54 32	31 00	1,783 81	4 00	300	5 00	11 00	1 25	18,000	Channel in rock	36 000	0 750	367 000	365 000	4 00	1 02,754	4,634	3,672	4 078	1,15,138	314	6	
7	Matoba Tank, ,,	10 00	15 28 / 4	32 00	6,095	48 41	29 00	1,763 64	9 00	600	3 00	9 00	1 50	10,000	Masonry wall	21 000	0 330	230 000	229 000	4 00	1,09,120	2,911	1,554	5,534	1,19,119	518	7	
8	Bhádalwádi Tank, ,,	23 00	22 91 / 4	23 00	2,590	55 09	35 00	1,629 56	6 00	550	5 00	11 00	1 33	20,000	Masonry wall and channel	15 000	0 313	223 000	222 000	4 00	96,536	3,616	5,432	4,190	1,09,774	492	8	550 feet is the originally designed length of the waste-weir and 400 feet is its actual length
9	Pashán Tank, ,,	16 00	*	41 00	2,750	52 00	21 00	1,911 98	6 00	400	4 00	10 00	1 12	11,000	Masonry wall	6 750	1 000	80 000	73 000	4 00	1 13,633	10,462	13,466	29,627	1,67,188	2,090	9	Town water-supply
10	Patas Tank, ,,	3 00	13 85 / 4	38 00	2,900	29 00	16 00	1,795 53	6 00	170	3 00	7 00	1 50	3,000	Masonry wall and channel	2 000	0 080	15 000	14 750	4 00	27,146	2,225	3,426	1,378	34,175	2,278	10	Do
11	Ekruk Tank, Sholapur	159 00	31 57 / 4	8 50	6,940	75 66	38 25	1,521 61	6 00	750	10 00	17 00	0 42	43,763	Two channels in excavation	198 232	12 000	3,330 000	3,310 000	7 00	5,45,205	64,406	1,07,144	61,522	7,78,277	234	11	
12	Ashti Tank, ,,	92 00	24 / 4	12 00	12,700	58 00	22 00	1 549 60	6 00	800	7 00	12 00	0 80	48,000	Excavated channel	123 296	23 000	1,550 000	1,348 000	4 00	3,80,065	16,433	10,765	57,545	4,61,808	299	12	
13	Pandharpur Tank, ,,	10 00	27 97 / 4	21 70	3,500	44 00	18 00	1,493 60	4 00	200	4 50	11 00	1 50	5,944	Excavated channel	8 530	2 000	89 000	79 000	5 00	85,652	4,753	16,837	431	1,07,673	1,210	13	Town water-supply
14	Mhasvád Tank, ,,	508 00	22 83 / 10	12 00	7 950	79 79	24 00	1,939 70	8 00	3,000	6 00	13 00	0 73	2,35,545	Concrete wall faced with masonry	174 840	48 500	3,072 130	2,632 770	4 00	7,72,748	28,392	63,481	78,800	9,43,421	307	14	
15	Nehr Tank, Satára	59 50	28 31 / 4	25 26	4,820	74 00	31 00	2,634 02	8 00	700	6 30	14 00	1 00	38 720	Wall and channel	29 490	3 400	522 640	489 770	4 00	2,63,213	8,645	33,687	38,141	3,43,686	658	15	Storage for Yerla Canals
16	Pingli Tank, ,,	20 00	23 05 / 4	33 11	5,553	53 50	29 00	2,342 09	6 00	750	3 00	9 00	1 00	12,862	Concrete wall faced with masonry and excavated channel	15 220	1 120	200 860	195 240	5 00	1,47,936	8,073	16,198	5,026	1 77,233	882	16	Do Gondoli Canal
17	Maini Tank, ,,	54 00	24 24 / 4	29 25	3,370	57 33	31 20	2,269 32	5 00	600 / 300	6 00	13 00	1 11	38,668	Masonry wall and channel and flood channel	16 570	1 200	195 270	188 590	5 00	1,53,066	14,869	40,527	3,354	2,11,816	1,085	17	
18	Islámpur Tank, ,,	2 45	28 57 / 4	46 87	2,892	30 97	15 00	Not available	6 00	200	2 00	7 00	0 82	1,305	Ditto	3 365	0 245	25 230	24 500	4 00	33,316	637	1,521	7,882	43,356	1,718	18	Town water-supply
19	Khás Tank, ,,	1 97	139 67 / 3	57 85	718	56 41	17 80	3,669 66	10 00	60	5 00	15 30	3 00	3,820	Masonry wall and excavated channel	3 373	0 333	—	56 567	4 00	50,998	11,110	12,102	2,523	76,733	1,355	19	Water-supply for Satára, small scheme abandoned for larger
20	Medleri Tank, Dhárwár	11 00	23 40 / 3	33 50	2,250	41 00	15 00	165' or 35' below datum assumed	6 00	700	2 00	7 00	1 00	6,453	Masonry wall and excavation	7 360	1 120	62 380	57 600	4 00	34,978	10,409	1,888	69	47,344	759	20	

[1] Extracted from a Return published by the Government of Bombay
[2] R L of sill above zero of local bench mark
* Information not available at present

In Column 4, the denominator represents the fractional part which the run off bears to the whole rainfall, thus 4 represents a run off = ¼ the whole rainfall
In Column 5, the fall of the river is that within the limits of the full supply contour, the distance being measured along the centre line of the bed of the river.

APPENDIX 2.

TABLE OF TOTAL MONSOON RAINFALL AND ESTIMATED RUN-OFF AND YIELD PER SQUARE MILE FROM CATCHMENT AREAS.

(*Vide* Chapter I., paragraphs 13 and 16, pages 20, 25.)

1	2	3	4	5	6	7	8	9	10
	Good Catchment			Average Catchment			Bad Catchment		
Total Monsoon Rainfall in inches.	Percentage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. cft.	Percentage of Run-off to Rainfall	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. cft.	Percentage of Run-off to Rainfall	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. cft.
1	0·1	0·001	0·002	0·1	0·001	0·001	0·05	0·0005	0·000
2	0·2	0·004	0·009	0·15	0·003	0·006	0·1	0·002	0·004
3	0·4	0·012	0·028	0·3	0·009	0·021	0·2	0·006	0·014
4	0·7	0·028	0·065	0·5	0·021	0·048	0·3	0·014	0·032
5	1·0	0·050	0·116	0·7	0·037	0·087	0·5	0·025	0·058
6	1·5	0·090	0·209	1·1	0·067	0·156	0·7	0·045	0·104
7	2·1	0·147	0·341	1·5	0·110	0·255	1·0	0·073	0·170
8	2·8	0·224	0·520	2·1	0·168	0·390	1·4	0·112	0·260
9	3·5	0·315	0·732	2·6	0·236	0·540	1·7	0·157	0·366
10	4·3	0·430	0·999	3·2	0·322	0·749	2·1	0·215	0·499
11	5·2	0·572	1·329	3·9	0·429	0·996	2·6	0·286	0·664
12	6·2	0·744	1·728	4·6	0·558	1·296	3·1	0·372	0·864
13	7·2	0·936	2·174	5·4	0·702	1·630	3·6	0·468	1·087
14	8·3	1·162	2·699	6·2	0·871	2·024	4·1	0·581	1·349
15	9·4	1·410	3·276	7·0	1·057	2·457	4·7	0·705	1·638
16	10·5	1·680	3·903	7·8	1·260	2·927	5·2	0·840	1·951
17	11·6	1·972	4·581	8·7	1·479	3·435	5·8	0·986	2·290
18	12·8	2·304	5·353	9·6	1·728	4·014	6·4	1·152	2·676
19	13·9	2·641	6·135	10·4	1·980	4·601	6·9	1·320	3·067
20	15·0	3·000	6·970	11·25	2·250	5·227	7·5	1·500	3·485
21	16·1	3·381	7·855	12·0	2·535	5·891	8·0	1·690	3·927
22	17·3	3·806	8·842	12·9	2·854	6·631	8·6	1·903	4·421
23	18·4	4·232	9·832	13·8	3·174	7·374	9·2	2·116	4·916
24	19·5	4·680	10·873	14·6	3·510	8·154	9·7	2·340	5·436
25	20·6	5·150	11·964	15·4	3·862	8·973	10·3	2·575	5·982
26	21·8	5·668	13·168	16·3	4·251	9·876	10·9	2·834	6·584
27	22·9	6·183	14·364	17·1	4·637	10·773	11·4	3·091	7·182
28	24·0	6·720	15·612	18·0	5·040	11·709	12·0	3·360	7·806
29	25·1	7·279	16·911	18·8	5·459	12·683	12·5	3·639	8·455
30	26·3	7·890	18·330	19·7	5·917	13·747	13·1	3·945	9·165
31	27·4	8·494	19·733	20·5	6·370	14·799	13·7	4·247	9·866
32	28·5	9·120	21·188	21·8	6·840	15·891	14·2	4·560	10·594
33	29·6	9·768	22·693	22·2	7·326	17·019	14·8	4·884	11·346
34	30·8	10·472	24·329	23·1	7·854	18·246	15·4	5·236	12·164
35	31·9	11·165	25·939	23·9	8·373	19·454	15·9	5·582	12·969
36	33·0	11·880	27·600	24·7	8·910	20·700	16·5	5·940	13·800
37	34·1	12·617	29·312	25·5	9·462	21·984	17·0	6·308	14·656
38	35·3	13·414	31·163	26·4	10·060	23·372	17·6	6·707	15·581
39	36·4	14·196	32·980	27·3	10·647	24·735	18·2	7·098	16·490
40	37·5	15·000	34·848	28·1	11·250	26·136	18·7	7·500	17·424
41	38·6	15·826	36·767	28·9	11·869	27·575	19·3	7·913	18·383
42	39·8	16·716	38·835	29·8	12·537	29·126	19·9	8·358	19·417
43	40·9	17·587	40·858	30·6	13·190	30·648	20·4	8·793	20·429
44	42·0	18·480	42·933	31·5	13·860	32·196	21·0	9·240	21·466
45	43·1	19·395	45·058	32·3	14·546	33·768	21·5	9·697	22·529
46	44·3	20·378	47·342	33·2	15·288	35·508	22·1	10·189	23·671
47	45·4	21·338	49·572	34·0	16·003	37·179	22·7	10·669	24·786
48	46·5	22·320	51·854	34·8	16·740	38·890	23·2	11·160	25·927
49	47·6	23·324	54·186	35·7	17·493	40·639	23·8	11·662	27·093
50	48·8	24·400	56·686	36·6	18·300	42·514	24·4	12·200	28·343
51	49·9	25·449	59·123	37·4	19·086	44·342	24·9	12·724	29·561
52	51·0	26·520	61·611	38·2	19·890	46·208	25·5	13·260	30·805
53	52·1	27·613	64·151	39·0	20·709	48·118	26·0	13·806	32·075
54	53·3	28·782	66·866	39·9	21·586	50·140	26·6	14·391	33·433
55	54·4	29·920	69·510	40·8	22·440	52·132	27·2	14·960	34·755
56	55·5	31·080	72·205	41·6	23·310	54·153	27·7	15·540	36·102
57	56·6	32·262	74·951	42·4	24·196	56·213	28·3	16·131	37·475
58	57·8	33·524	77·883	43·3	25·148	58·412	28·9	16·762	38·941
59	58·9	34·751	80·734	44·1	26·063	60·550	29·4	17·375	40·367
60	60·0	36·000	83·635	45·0	27·000	62·726	30·0	18·000	41·817

NOTE.—Plate 1 shows these results diagrammatically.

APPENDIX 3.

CALCULATION OF THE AMOUNT OF STORAGE REQUIRED FOR THE IRRIGATION OF A CERTAIN AREA AND TO SUPPLEMENT THE DISCHARGE OF A NATURAL STREAM.

(*Vide* Chapter I , paragraph 26, page 45.)

1. The following calculation shows how the amount of storage which is required to supplement the discharge of a natural stream may be determined The stream considered is one which has a deficient supply in the fair season, and a superabundant one during the monsoon By storing only sufficient of the excess discharge during the latter period to tide over the deficiency of the former one, the catchment can be made to serve the area contemplated at the minimum expense, as the normal daily discharge of the stream will thus be fully utilized.

2. The first thing to be done is to calculate the daily supply required by the canal and then to gauge the daily discharge of the stream, as the irregularity of the flow of the latter will not permit of monthly results being taken directly into account. When the discharge of the stream is in excess of the requirements of the canal, the balance is available for being stored. When, however, the former is less than the latter, the difference will have to be supplied by the storage already effected.

3 The daily observations should be recorded in the following form :—

REGISTER OF DAILY GAUGINGS.

1	2	3	4	5
Date.	Discharge required for Canal Average for day. Cft per sec	Discharge of Natural Stream at Weir Site Average for day. Cft per sec	Discharge required from Storage. Average for day. Cft. per sec	Discharge that can be Stored Average for day. Cft per sec

At the end of the month the columns should be totalled and multiplied by 86,400, the number of seconds in one day, and the results thus obtained tabulated in the form given below :—

Estimate of Storage Required

1	2	3	4	5	6
Year and Month	Quantity required for Canal Mill cft	Discharge of Natural Stream at Weir Site Mill cft	Quantity required from Storage Mill. cft	Quantity that can be Stored Mill. cft	Contents in Reservoir, with F S Storage = 150 mill cft (at end of each month) Mill. cft
			Initial	storage	130 373
January	26 784	5 575	21·209	nil	109 164
February .	25 056	4·150	20·906	nil	88·258
March	26 784	1 584	25·200	nil	63 058
April	25·920	10·146	22·397	6 623	47·284
May .	26 784	0·453	26·331	nil	20 953
June .	39·744	25 871	32 587	18·714	7 080
July .	53 568	19·982	40·576	6·990	**26 506**
August	53 568	35 717	39 359	21·508	**44·357**
September	51·840	10 262	41·578	nil	**85·935**
October .	38 880	68 947	25·799	55 866	30·067
November .	25·920	4 881	21 039	nil	9·028
December .	26·784	16 045	20 876	10·137	**1·711**
Totals .	421·632	203·613	337 857	119·838	

Notes.

1. Col. 2 + col 5 = col. 3 + col. 4.

2. The increments in col. 6 are equal to the excesses of col 5 over col. 4, and the decreases, to those of col. 4 over col. 5.

3. In col. 6 the entries in light type indicate the contents at the ends of the months of a reservoir with an assumed F.S. Storage of 150 mill. cft., no allowance for evaporation, etc., being made. The entries in heavy type indicate the additional storage required in the reservoir to enable it to tide over the periods concerned.

		Mill. cft.
4	The total of col. 3	203·613
	Plus the initial storage	130 373
	Minus the final storage (in this case a minus quantity) . .	**1·711**
	Plus the maximum deficiency	85·935
	Equals the total of col. 2	421·632

5. To ascertain the amount of storage required .—to the assumed storage add the maximum deficiency, or deduct the minimum excess, at the beginning of the rains, and then add the proper allowances for evaporation and absorption and for loss in transit down the feed channel.

APPENDIX 4.

STORAGE EXPENDITURE ESTIMATE.

(*Vide* Chapter I., paragraph 27, page 46.)

1. For the proper regulation of the draw-off from a reservoir it is necessary to frame an estimate of the expenditure of water, month by month. The following data may be assumed :—

2. Duty of Water.—This may be taken thus :—

Nov. 1st—Feb. 28th	.	60 acres per cft per sec.
March 1st—June 30th	.	40 ,, ,, ,,
July 1st—Oct 31st	.	80 ,, ,, ,,

Note 1.—The actual seasons begin and end fifteen days earlier than these dates, but this may be neglected for the estimate.

Note 2.—The duties are purposely taken low so as to agree with what may occur in practice ; by good management it should be easy to increase them.

Note 3.—The duties assumed are average ones for all classes of crops that may be irrigated at the same time.

3. Acreage under Irrigation.—It will be necessary to assume the acreage under irrigation from time to time, and the utility of the estimate will depend upon the correctness of the assumptions made The acreage assumed should be as large as previous experience indicates, but subject to the restriction noted in paragraph 7 below.

4. Amount of Draw-off.—This should be calculated on the acreage assumed and on the duties given in paragraph 2 above.

5. Allowance for Evaporation and Absorption, &c.—This may be assumed thus (para. 2) .—

Nov. 1st—Feb. 28th	. .	3 in depth per month over the then top area of the reservoir.
March 1st—June 30th	. .	8 in ,, ,, ,,
July 1st—Oct. 31st	. .	4 in. ,, ,, ,,

These amounts should be deducted from the estimated reservoir level at the end of each month so as to give the estimated level at the beginning of the next month. They correspond to a loss of storage during the year which is equal to a draw-off of 5 feet in vertical depth from the mean area of the reservoir.

6. Available Supply.—The amounts available at the different reservoir levels should be taken from the table of reservoir contour capacities (Appx. 17, p. 382) The necessary deductions for the silting-up of the reservoir should be made.

7. Extent of Estimate.—The estimate should extend from the time when there is no more chance of replenishment, say November 1st, until there is a fair certainty of replenishment during the next monsoon, as ascertainable from the records. It should start with the actual level of the reservoir. The estimated future levels, when once sanctioned, should not be exceeded without further sanction, which should be obtained after giving full explanation of the causes necessitating it. The estimate should close with a small balance in the reservoir to allow for contingencies, and, for the same reason, credit should not be taken for hot-weather replenishments. It should be submitted on November 15th

8. Type Estimate.—The following is given as part of a type estimate for a reservoir supplying a canal which is also partly fed by a river .—

Month.	At beginning of month: Estimated R. L. of Reservoir	At beginning of month: Storage in Reservoir Mill. cubic feet.	River Supply. Million cubic feet	Total available supply. Million cubic feet.	Acres irrigated.	Duty allowed, acres per cubic foot per second.	Total consumption of water. Million cubic feet	Consumption of water from Reservoir alone Million cubic feet.	R L of Reservoir due to consumption	Allowance for evaporation, etc. Feet	Estimated R L. of Reservoir at end of month.	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13
Nov	113·25	247·86	134·14	382·00	2,000	60	86·400	0·00	113·25	0·25	113·00	
Dec.	113·00	242·29	57·71	300·00	2,500	60	111·600	53·89	109·51	0·25	109·26	

APPENDIX 5.

ESTIMATE OF THE DURATION OF A WATER-SUPPLY STORAGE.

(*Vide* Chapter I., paragraph 27, page 46.)

The following estimate was made of the duration of supply in a connected series of reservoirs designed for the water-supply of the town of Dharwar, Bombay Presidency. It was assumed in it that the storage of the upper reservoirs would first be utilised, being run-off for this purpose into the main Kelgeri Storage Reservoir, and that the latter would be the last to be drawn upon. For the sake of simplicity of calculation it was further assumed that the storage required to make good the loss by evaporation, etc., from the lower tanks was not replaced from the upper subsidiary tanks.

1	2	3	4	5	6	7	8	9	10	11
Month	At commencement of month. Tank			Evaporation, etc		Draw-off	Total deplenishment	At end of month Tank.		
	R.L	Surface area.	Contents.	Depth on top area	Amount			Contents	R.L	Surface area.
		Mill sq feet	Mill. cft.	Feet	Mill cft	Mill cft.	Mill cft	Mill. cft		Mill. sq feet
					NAIKANKHEDI TANK					
Nov.	F S L 91·00	0 649	2 528	0·16	0·104	2·100	2·528	Nil, Tank run dry on Nov. 30th.		
	Balance in tank and allowed for contingencies . . .				0·324					
					KENCHANHATTI TANK.					
Nov	F S.L. 80·00	1·220	4·879	0·16	0·195	—	0 195	4·684	79·80	1·200
Dec. ..	79·80	1·200	4·684	0 16	0 192	2 170	2 362	2·322	79·50	0·775
Jan.	79·50	0·775	2 322	0·16	0·124	2·170	2 322	Nil, Tank run dry on Jan. 31st.		
	Balance in tank and allowed for contingencies . . .				0 028					

1	2	3	4	5	6	7	8	9	10	11
Month.	At commencement of month			Evaporation, etc.		Draw-off	Total depleishment	At end of month.		
	Tank			Depth on top area	Amount.			Tank.		
	R.L	Surface area	Contents					Contents	R.L	Surface area.
		Mill sq feet	Mill. cft	Feet	Mill. cft	Mill. cft.	Mill. cft.	Mill. cft.		Mill. sq feet
Kelgeri Urmundinkeri.										
Nov.	F S L 78·76	2·932	12·106	0·16	0·469	—	0·469	11·637	78·60	2·870
Dec.	78·60	2 870	11·637	0 16	0·459	—	0·459	11·178	78·44	2·790
Jan.	78 44	2 790	11 178	0 16	0·446	—	0·446	10·732	78·28	2 730
Feb	78·28	2 730	10·732	0 33	0 910	1 960	2·870	7·862	77·15	2·320
March	77 15	2·320	7 862	0·67	1·554	2 170	3·724	4·138	75·25	1·680
	75 25	1·680	4 138	0 67	1·120					
April	Balance in tank and allowed for contingencies . . .				0·918	2·100	4·138	Nil, Tank not drawn on after April 30th.		
Kelgeri Storage Reservoir.										
Nov.	F.S L. 60 00	6·599	59·299	0 16	1·056	—	1·056	58·243	59·84	6·530
Dec.	59·84	6 530	58·243	0 16	1·045	—	1·045	57·198	59·68	6·470
Jan.	59 68	6 470	57 198	0 16	1·035	—	1·035	56·163	59 52	6·400
Feb	59 52	6·400	56·163	0 33	2·133	—	2·133	54·030	59 19	6·280
March	59·19	6·280	54·030	0 67	4·120	—	4·120	49 910	58·52	6 000
April	58 52	6·000	49·910	0·67	4·000	—	4·000	45·910	57·85	5·770
May	57·85	5 770	45·910	0 51	2·942	2·170	5·112	40·798	56·90	5·420
June	56·90	5·420	40·798	0·31	1·807	2 100	3·907	36·891	56·16	5·154
July	56 16	5·154	36·891	0 16	0·825	2·170	2·995	33·896	55·50	4·935
Following June	55 50	4 935	33·896	3·48	11·091	23·360	34·471	Nil, Tank dry June 30th.		
At pumping sill		1·437		During 11 months.						
Mean area		3·187								

Notes.

1. These calculations show that, starting with full tanks on November 1st, the supply would last up to June 30th of the next year but one, and that they would thus tide over a year during which there was no replenishment.

2. The allowances in feet for evaporation and absorption, on the *top areas*, were :—

	Nov.	Dec.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.
For	0 16	0·16	0·16	0·33	0 67	0·67	0·51	0·31	0·16	0·16	0·16	0·17

i.e., for the whole year 3·65 feet. A separate allowance was not estimated for absorption, as it was believed it would be very small, for the beds of all of the tanks are formed of puddled rice fields. On the other hand, full credit was not given for the rain falling on the tanks themselves, which would compensate for a good deal of loss on this account.

3. The daily supply to be given to the town is 430,000 gallons, or 68,900 cubic feet. In the calculations it has been taken equal to 70,000 cubic feet.

APPENDIX 6.

STATISTICS OF CERTAIN IRRIGATION WORKS IN INDIA.

(*Vide* Chapter I., paragraphs 29—31, pages 49–52.)

TABLE I.

DUTY OF WATER OBTAINED ON CERTAIN IRRIGATION WORKS IN INDIA DURING 1885—86, 1890—91, AND 1895—96.

(*Vide* Irrigation Revenue Reports, Statistical Table I.—E.)

Province.	Name of Work	Acres irrigated per cubic foot of discharge utilised.					
		1885–86		1890–91.		1895–96.	
		Kharif	Rabi	Kharif	Rabi	Kharif.	Rabi
1	2	3	4	5	6	7	8
	1 TANKS						
Bombay	1 Mhasvád Tank	148·00	43·00	88·54	52·41	41·64	41·21
	2 Ekruk Tank	50 83	28 07	57 77	29 38	60 76	31·05
United Provinces	3. Jhansi Lakes	—	—	—	21·00	7·00	72·00
	4 Hamirpur Lakes	—	29·00	—	31·00	7·00	37·00
Rajputana	5 Bir Tank	—	—	—	—	10·00	36 00
	6. Ladpura Tank	—	—	—	—	49 00	51·00
	7 Balad old and new Tanks	—	—	—	—	18·00	22·00
Baluchistan	8 Khushdil Khan Reservoir	—	—	—	—	—	138·66
	2 CANALS.						
Bombay	9. Nira Canal	115 00	60·00	76·24	56·29	54·09	54·22
	10. Hathmati Canal	16 20	47·44	40·84	67·56	36 86	54·00
	11 Krishná Canal	61 00	22·00	38·00	37·00	42·00	25·00
Sind	12 Mithrau Canal	51·55		34·37		37·58	28·37
	13 Sukkur Canal	29·85		83·08		37·17	240·17
United Provinces	14. Upper Ganges Canal	86·00	171·00	81·00	142·00	66·00	123·00
	15 Agra Canal	97 00	131·00	95·00	115·00	68·00	105·00
	16. Betwa Canal	—	65·00	28·00	77·00	28·00	90·00
Punjab	17 Swat River Canal	—	158·00	57 00	126·00	77·00	116·00
	18. Bari Doab Canal	64·00	157·00	64·00	140·00	68·00	135·00
	19. Sirhind Canal	102 00	193·00	66·00	169·00	44·00	156·00
Bengal	20. Orissa Canals	80·10	—	90·99	11·32	67·00	—
	21. Sone Canals	94·30	99·90	88·48	47·41	74·32	72·87
Baluchistan	22. Shebo Canal	—	—	35·00	94·00	20·80	99·70
Madras	23. Godavari Delta System	90 45	7·04	94·93[1]	8·88[2]	94·28[1]	27·82[2]
	24 Cauvery "	71·34	58·37	72·21[1]	1204·61[2]	61 94[1]	90·73[2]
	25. Kistna "	78·24	—	124·44[1]	—	125·05[1]	—

[1] First crop. [2] Second crop.

For Sind and Madras the figures given are the areas irrigated per cubic foot of discharge at head.

Table II.

Duty of Water obtained on certain Irrigation Works in Bombay (Deccán), giving Average Results.

(*Vide* Irrigation Revenue Report, Statistical Table I.—E.)

Name of Work	Utilised Discharge.													
	Kharif Duty.							Rabi Duty.						
	1894–95	1895–96	1897–98	1898–99	1900–01	Total	Average	1894–95	1895–96	1897–98	1898–99	1900–01	Total	Average.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1. Mhasvád Tank	34	42	13	73	109	271	54	50	41	101	42	51	285	57
2. Ekruk Tank	61	61	86	83	210	501	100	38	31	44	44	—	157	89
3. Bhátodi Tank	122	19	118	14	82	355	71	135	123	154	161	62	635	127
4. Nirá Canal	48	54	100	85	153	440	88	57	54	124	91	118	444	89
5. Krishná Canal	49	42	52	38	137	318	64	22	25	26	26	53	152	30
6. Lower Pánjhrá Canals	103	94	77	375	—	649	162	—	132	—	—	142	274	137
Totals	417	312	446	668	691	2,534	539	302	406	449	364	426	1,947	479
Average	70	52	74	111	138	445	89	60	68	90	75	85	378	76

The years 1896–97 and 1899–1900 are excluded, being famine years.

Table III.

Proportions of Areas Irrigated to Culturable Areas under Command on Irrigation Works in Bombay (Deccán).

(*Vide* Irrigation Revenue Reports, Statistical Table IV.—E.)

Year.	Culturable area in acres.	Irrigated area in acres.	Percentage of col 3 to col 2	Remarks.
1	2	3	4	5
1888–89	512,301	79,195		The entries in col. 2 for the years 1888–89 to 1893–94 are believed to show the total culturable area under command of the completed projects (*vide* Irrigation Revenue Report, 1894–95, para. 6 of Govt. Resolution, No 9, W I, 123 of Jan. 23rd, 1896). For the remaining years the entries in col 2 show the culturable area under command of the projects as actually constructed.
1889–90	533,313	80,599		
1890–91	535,762	75,901		
Total	1,581,376	241,695	15·3	
1891–92	559,911	97,074		
1892–93	571,903	66,486		
1893–94	583,897	81,000		
Total	1,715, 711	244,560	14·3	
Grand Total, 1888–1894	3,297,087	486,255	14·7	
1894–95	341,015	85,394		
1895–96	315,040	76,129		
1896–97	279,741	119,210		
Total	935,796	280,733	30 0	
1897–98	316,425	127,722		
1898–99	316,425	105,055		
1899–1900	316,425	105,830		
Total	949,275	338,607	35·7	
Grand Total, 1894–1900	1,885,071	619,340	32 9	
1900–01	316,425	126,178	39·9	
Grand Total, 1894–1901	2,201,496	745,518	33·9	

Table IV.

Areas Irrigated under Different Crops in Bombay (Deccán).

(*Vide* Irrigation Revenue Reports, Statistical Table III.—E.)

Description of Crop.		Acres Irrigated.						
		1894–95	1895–96	1897–98	1898–99	1900–01	Total.	Aver.
1		2	3	4	5	6	7	8
1. Perennial	Area	*17,633	14,533	10,198	11,094	10,766	64,224	12,845
	Percentage	22	20	8	11	9	13	13
2 Rabi	Area	*14,823	*12,035	21,023	25,120	21,521	94,522	18,904
	Percentage	18	17	17	25	19	19	19
3. Monsoon dry	Area	34,788	33,459	*75,198	48,812	*71,167	263,424	52,685
	Percentage	43	46	62	49	61	45	54
4. Eight months	Area	12,303	10,596	11,702	12,762	12,196	59,649	11,930
	Percentage	15	15	10	13	10	12	12
5 Hot weather	Area	1,117	1,101	3,332	1,538	1,783	8,871	1,774
	Percentage	2	2	3	2	1	2	2
6. Total	Area	80,754	71,724	121,453	99,326	117,433	490,690	98,138
	Percentage	100	100	100	100	100	100	100

Table IVa.

Areas and Percentages Omitting Exceptional Areas, marked thus * in Table IV.

Description of Crop.	Acres Irrigated.		
	Total.	Average.	Percentage.
1	2	3	4
1. Perennial	46,591	11,658	15
2. Rabi	67,664	22,555	23
3. Monsoon dry	117,059	39,020	40
4. Eight months	59,649	11,930	20
5. Hot weather	5,539	1,385	2
6. Grand total	296,502	—	100

The years 1896–97 and 1889–1900 are excluded, being famine years.

APPENDIX 7.

(*Vide* Chapter I., paragraphs 28, 29 and 32, pages 47, 49, 52.)

ESTIMATES OF REVENUE RESULTS FROM RESERVOIR STORAGE.

1. Estimate of the Irrigating Capacity of Reservoir Storage.

Let A = the area in acres of each crop in quadrennial rotation ;

Q = the quantity in cubic feet required for the irrigation of 4 A ;

The duty for perennial crops (sugar, plantains, &c.) = 100 acres for 365 days ;

The duty for rice = 40 acres for 122 days ;

The duty for monsoon dry crops = 160 acres for 122 days ,

The duty for rabi = 120 acres for 121 days

	Discharge cft. per sec.		Secs.		Days.		Cubic feet.
Then Q =	$\frac{A}{100}$	$\times$	86,400	$\times$	365	=	315,360A
+	$\frac{A}{40}$	$\times$	86,400	$\times$	122	=	263,520A
+	$\frac{A}{160}$	$\times$	86,400	$\times$	122	=	65,880A
+	$\frac{A}{120}$	$\times$	86,400	$\times$	121	=	87,120A

Thus Q for 4 A . . . = 731,880A, say 750,000A, (cft.)

adding one-third for loss by evaporation and absorption = E.

$$Q + E \text{ for } 4\ A = 1{,}000{,}000A \text{ cubic feet.}$$

Or, in other words, if A = 1 acre, with these duties the reservoir will irrigate 4 acres per million cubic feet of storage.

If the duties are taken three-quarters of the above, the reservoir will irrigate 3 acres per million cubic feet of storage.

If the duties are taken one-half of the above, the reservoir will irrigate 2 acres per million cubic feet of storage.

2. Estimate of the Return from Reservoir Storage.

The following table shows the return which may be expected from the storage of one million cubic feet, assuming :—

That the quoted rates are assessed for the irrigation of the crops :

(1) when the work is first opened, and

(2) after it has been in operation for some years ;

that working expenses are at the rate of Rs. 2 per acre ;

and that, respectively, full, three-quarter and half duties (as per Estimate above) are obtained. Applying the rates in force in any

Estimate of Revenue from a Storage of One Million Cubic Feet.

		2	3	4	5	6	7	8	9
	Crop.	At Original Rates				At Final Rates.			
		Rate per acre	Revenue			Rate per acre.	Revenue.		
			Full duty	¾ duty.	½ duty.		Full duty.	¾ duty.	½ duty.
		Rs	Rs.	Rs	Rs	Rs.	Rs.	Rs	Rs.
1	Perennial	10	10·00	7·50	5·00	16	16 00	12 00	8·00
2.	Rice	4	4·00	3·00	2 00	6	6·00	4·50	3·00
3	Monsoon dry	1	1·00	0·75	0 50	1	1·00	0·75	0 50
4	Rabi	3	3 00	2 25	1·50	4	4·00	3·00	2·00
5.	Gross revenue	—	18 00	13·50	9·00	—	27·00	20·25	13 50
6	Working expenses.	2	8 00	6·00	4·00	2	8 00	6·00	4·00
7.	Net Revenue	—	10 00	7·50	5·00	—	19·00	14 25	9 50

locality in this manner, it can be seen what will be the probable revenue from the storage ; and, estimating the approximate cost of the storage, the approximate return from the capital expenditure can be obtained.

3. Estimated Amount of Storage for Different Classes of Crops per Rupee of Assessment.

The following table shows the estimated amount of storage per rupee of assessment required for the different crops. This is deduced from :—

(*a*) The estimated consumption of water utilised for bringing them to maturity, plus the estimated loss by evaporation which occurs from the storage until they are matured ; and

(*b*) the crop rates :—

ESTIMATED AMOUNT OF STORAGE PER RUPEE OF ASSESSMENT.

1	2	3	4	5	6	7	8	9	10
Crop.	Quantity of Storage Required					At Original Rates		At Final Rates.	
	Evaporation			Storage utilised	Total	Assessment per acre	Amount of Storage per rupee of Assessment.	Assessment per acre.	Amount of Storage per rupee of Assessment.
	Loss until crop is matured	Percentage loss	Amount of loss						
	Inches		c ft.	c ft	c ft.	Rs.	c ft	Rs.	c. ft
1. Perennial .	48	46	115,000	325,000	440,000	10	44,000	16	27,500
2. Rice .	16	16	40,000	270,000	310,000	4	77,500	6	51,667
3. Monsoon dry	16	16	40,000	65,000	105,000	1	105,000	1	105,000
4. Rabi .	24	22	55,000	90,000	145,000	3	48,333	4	36,250
5 Totals .	48 (during year)	100	250,000	750,000	1,000,000	—	—	—	—

The assessments are usually fixed with reference to the value of the crops, and not according to the amount of water which has to be stored for them. The above table shows, on the assumptions made, that under reservoirs the irrigation of valuable perennial crops should be encouraged, while that of monsoon crops should be discouraged so far as revenue and return are concerned. Unless, however, the land is heavily manured, a rotation of crops is necessary, and the return on the capital expenditure must therefore be estimated on the average results from the different crops under cultivation.

APPENDIX 8.

TABLE OF WASTE-WEIR RUNS-OFF.

(*Vide* Chapter III., paragraph 171, page 230.)

1	2	3	4	5	6
Increments of Catchment Area.	Run-off from each increment of Catchment Area in col. 1.	Discharge from each increment of Catchment Area in col. 1 due to Run-off	Discharge from Total Catchment Area due to Run-off.	Average Run-off from Total Catchment Area.	Remarks.
Square miles.	Inches per hour	Cubic feet per second.	Cubic feet per second	Inches. per hour.	
0—1	3·00	1,936	1,936	3·00	
1—2	**2 64**	**1,704**	**3,640**	**2·82**	
2—3	2·30	1,490	5,130	2·65	
3—4	**2·00**	**1,291**	**6,421**	**2·49**	
4—5	1·85	1,194	7,615	2·36	
5—6	**1·72**	**1,110**	**8,725**	**2·25**	
6—7	1·62	1,045	9,770	2·16	
7—8	**1·52**	**981**	**10,751**	**2·08**	
8—9	1·45	936	11,687	2·01	
9—10	**1·40**	**903**	**12,590**	**1·95**	
10—15	1·16	3,743	16,333	1·69	
15—20	**1·00**	**3,227**	**19,560**	**1·51**	
20—25	0·92	2,968	22,528	1·40	
25—50	**0·80**	**12,907**	**35,435**	**1·10**	
50—75	0·72	11,565	47,000	0·97	
75—100	**0·65**	**10,500**	**57,500**	**0·89**	
100—150	0·60	19,500	77,000	0·80	
150—200	**0·53**	**17,000**	**94,000**	**0·73**	

Col. 3 = Increment of area in Col. 1 × Col. 2 × 645·33 cubic feet.
Col. 4 = Sum of entries in Col. 3.

$$\text{Col. 5} = \frac{\text{Col. 4}}{\text{Last area in Col. 1} \times 645{\cdot}33.}$$

Note.—Plate 2 shows the entries in Cols. 2 and 5 diagrammatically.

APPENDIX J.

TABLES OF VALUES OF BAZIN'S COEFFICIENTS.[1]

(*Vide* Chapter III., paragraph 172, page 237.)

Table I.—For Earthen Channels.

r.	c_2.	Mean vel. / Max. vel.	r.	c_2.	Mean vel. / Max. vel.	r.	c_2.	Mean vel. / Max. vel.
0 05	11·9	·320	2·65	67 8	728	6·5	84·8	·770
0·10	16·7	·398	2·70	68 2	·729	6·6	85·0	·771
0·15	20·3	·446	2·75	68·6	·731	6 7	85·3	·771
0·20	23·4	·480	2·80	69 0	·732	6 8	85·5	·772
0·25	25·9	·506	2·85	69·3	·733	6·9	85·7	·772
0·30	28·3	·528	2 90	69·7	·734	7 0	86 0	·773
0·35	30·4	·545	2 95	70 0	·735	7·1	86 2	·773
0 40	32·3	·561	3·00	70·4	736	7·2	86·4	·774
0·45	34 0	·574	3 05	70·7	·737	7 3	86·6	·774
0·50	35·7	·585	3·10	71·0	·737	7 4	86 8	·774
0·55	37·2	·595	3·15	71 4	·738	7 5	87 0	·775
0 60	38·7	·605	3 20	71·7	·739	7·6	87·2	·775
0 65	40 0	·613	3 25	72 0	·740	7 7	87·4	·776
0 70	41·3	620	3 30	72 3	·741	7·8	87·6	·776
0 75	42·6	·627	3 35	72 6	·741	7·9	87·8	·776
0 80	43·7	·634	3·40	72 9	·742	8·0	88 0	·777
0·85	44·9	·639	3·45	73·2	·743	8·2	88 4	·778
0·90	45·9	·645	3 50	73 5	·744	8·4	88·7	·778
0·95	46·9	650	3 55	73 7	·745	8 6	89·1	·779
1 00	47 9	·654	3 60	74 0	·745	8·8	89·4	·779
1 05	48·9	·659	3·65	74 3	·746	9 0	89 7	·780
1·10	49·8	·663	3·70	74 6	·747	9·2	90·0	·781
1·15	50 7	·667	3 75	74·8	·747	9 4	90 3	·781
1·20	51·5	·671	3 80	75 1	748	9·6	90·6	·782
1 25	52 3	674	3 85	75 3	·749	9 8	90·9	·782
1·30	53·1	677	3 90	75 6	·749	10 0	91·2	·783
1·35	53·9	·680	3 95	75·8	·750	10·2	91 4	·783
1 40	54·6	683	4·0	76 1	·750	10·4	91·7	·784
1·45	55·3	·686	4·1	76 5	·752	10·6	91·9	·784
1 50	56·0	·689	4·2	77·0	·753	10 8	92·2	·785
1·55	56·7	·691	4 3	77 5	·754	11·0	92 4	·785
1·60	57·4	·694	4·4	77·9	·755	11·2	92·6	·785
1·65	58·0	·696	4 5	78·3	·756	11·4	92·8	·786
1·70	58·6	·698	4 6	78·7	·757	11·6	93·1	·786
1·75	59 2	701	4·7	79·1	·758	11·8	93·3	·787
1·80	59·8	·703	4 8	79·5	·759	12 0	93 4	·787
1 85	60·4	·705	4·9	79·9	·760	13·0	94·4	·789
1·90	60·9	·707	5·0	80·2	·760	14·0	95·2	·791
1 95	61·5	·708	5 1	80·6	·761	15 0	95·9	·793
2 00	62 0	·710	5·2	80 9	·762	16 0	96·6	·795
2·05	62 5	·712	5·3	81 3	·763	17 0	97·2	·797
2·10	63 0	714	5·4	81·6	·763	18·0	97·7	·798
2·15	63 5	·715	5·5	81 9	764	19·0	98·2	·798
2·20	64·0	·717	5 6	82·3	·765	20 0	98 6	·800
2 25	64 4	·718	5·7	82·6	·765	25·0	100·3	·804
2 30	64 9	·720	5 8	82·9	·766	30·0	101·5	·806
2·35	65 3	·721	5 9	83·2	·767	35·0	102·4	·808
2·40	65·8	·722	6·0	83·4	·767	40·0	103·1	·809
2·45	66 2	·723	6·1	83·7	·768	50 0	104·1	·809
2·50	66·6	·725	6 2	84 0	·768	70·0	105·2	·810
2·55	67·0	726	6·3	84·3	·769	100·0	106·1	·810
2 60	67·4	·727	6·4	84·5	·770	Inf.	108·3	·810

[1] Extracted from Higham's "Hydraulic Tables", Nos. III and IV.

TABLE II.—FOR MASONRY CHANNELS.

COEFFICIENTS — c_2.

r.	Class I	Class II	Class III.	*r.*	Class I	Class II.	Class III
0·25	125 4	94 8	56·5	3 75	146 0	127·6	105·9
0·50	135 3	108 8	72 0	4·00	146 1	127·8	106·5
0·75	139 1	115 0	80 8	4 50	146	128	107
1·00	141·2	118·5	86 7	5·00	146	128	108
1·25	142·4	120 8	90 9	5 50	146	129	109
1·50	143 3	122·4	94 0	6 00	147	129	110
1·75	143 9	123 6	96 5	6·50	147	129	110
2·00	144·4	124·5	98 5	7 00	147	129	110
2·25	144 8	125 2	100 1	7·50	147	129	111
2·50	145 1	125·8	101 5	8·00	147	130	111
2·75	145 4	126 2	102·6	8·50	147	130	112
3 00	145·6	126·7	103 6	9 00	147	130	112
3 25	145·7	127 0	104·5	10 00	147	130	112
3 50	145 9	127 3	105 2	100·00	148	131	116

RATIOS OF MEAN TO MAXIMUM VELOCITY.

r.	Class I.	Class II	Class III.	*r.*	Class I.	Class II.	Class III
1·00	0·85	0 83	0·77	5·00	0 85	0·83	0·81
2 00	0·85	0·83	0·79	6·00	0 85	0·84	0·81
3·00	0 85	0·83	0·80	9·00	0 85	0·84	0·82
4·00	0·85	0 83	0·81	100 00	0·85	0 84	0·82

Class I.—Bed and sides, fine plastered, planed planks, &c.
,, II.—Bed and sides, cut stone, brickwork, planking, &c.
,, III.—Bed and sides, rubble masonry.

APPENDIX 10.

TABLE OF THE DISCHARGE OF A WASTE-WEIR CHANNEL HAVING A BED WIDTH OF 200 FEET AND A BED SLOPE OF 1 IN 100.

(*Vide* Chapter III, paragraph 174, page 239.)

1	2	3	4	5	6	7	8	9
Total Depth	Afflux Height, d_1.	Tail Depth, d_2.	Afflux Co-efficient, c_1.	Channel Co-efficient, c_2	$\frac{D}{c_1 b \sqrt{2g}}$	$\sqrt{d_1}\,(d_2 + \frac{2}{3}d_1)$.	Mean Velocity of Tail Channel.	Dis-charge, D.
Feet.	Feet.	Feet					Feet per second.	Cubic Feet per second.
1	0·30	0·70	0·60	41·00	0·495	0·495	3·40	476
2	**0·74**	**1·26**	**0·60**	**52·10**	**1·510**	**1·505**	**5·78**	**1,457**
3	1·24	1·76	0·60	59·00	2·85	2·87	7·78	2,741
4	**1·70**	**2·30**	**0·62**	**64·4**	**4·47**	**4·46**	**9·66**	**4,444**
5	2·15	2·85	0·64	68·8	6·34	6·29	11·42	6,510
6	**2·63**	**3·37**	**0·66**	**72·0**	**8·29**	**8·29**	**13·03**	**8,784**
7	3·08	3·92	0·68	75·0	10·46	10·45	14·55	11,407
8	**3·52**	**4·48**	**0·70**	**77·5**	**12·80**	**12·84**	**16·04**	**14,374**
9	3·95	5·05	0·72	79·5	15·23	15·28	17·41	17,585
10	**4·36**	**5·64**	**0·74**	**81·4**	**17·87**	**17·87**	**18·80**	**21,210**
11	4·77	6·23	0·76	83·1	20·56	20·51	20·11	25,057
12	**5·17**	**6·83**	**0·78**	**84·5**	**23·34**	**23·34**	**21·38**	**29,203**
13	5·54	7·46	0·80	85·8	26·24	26·20	22·57	33,668
14	**6·00**	**8·00**	**0·80**	**86·8**	**29·44**	**29·40**	**23·61**	**37,775**
15	6·47	8·53	0·80	87·7	32·65	32·61	24·56	41,893
16	**6·94**	**9·06**	**0·80**	**88·6**	**36·03**	**36·00**	**25·51**	**46,236**
17	7·42	9·58	0·80	89·3	39·47	39·52	26·43	50,645
18	**7·86**	**10·14**	**0·80**	**90·0**	**43·10**	**43·06**	**27·28**	**55,304**
19	8·33	10·67	0·80	90·7	46·88	46·81	28·19	60,157
20	**8·80**	**11·20**	**0·80**	**91·3**	**50·67**	**50·63**	**29·03**	**65,035**

NOTES.

1. In the Table the afflux coefficient c_1 has been taken approximately. Rankine[1] gives Poncelot and Lebros' coefficients, which were determined from experiments with sharp-edged rectangular orifices only about 8 inches wide in vertical flat plates, and it is believed none on a large scale have ever been made. The values of c_1 are increased up to a total flood-depth of 13 feet, on account of the more efficient discharging power of a deep channel. No further increase of them is thereafter made, as large waves, and consequently more friction, will be caused. Any errors made by these assumed values will be minimised in the calculations given in Appendices 12, 13, and 14, in the proportion that the discharges of the waste-weir cut bear to the total flood, and this will always be small.

2. The channel coefficient c_2 is found from Appendix 9 (I.) with reference to r therein, and not to d_2 of this Appendix. In tail channels of considerable width r is, however, only very slightly less than d_2.

[1] "Civil Engineering," 11th edn., art. 448, p. 680.

APPENDIX 11.

TABLE OF THE DISCHARGE FROM EACH FOOT OF LENGTH OF A CLEAR OVERFALL WEIR, IN CUBIC FEET PER SECOND.

(*Vide* Chapter III., paragraph 176, page 242.)

Calculated by the formula :—$D = \frac{2}{3} cbd\sqrt{2gd}$

Height of still water above weir crest $= d$.

CUBIC FEET PER SECOND.

Feet.	Decimals of a Foot									
	0	·1	·2	·3	4	·5	·6	·7	8	·9
0	0·00	0·10	0·28	0·52	0 81	1·14	1 51	1 91	2 34	2 80
1	3 29	3 81	4 35	4 91	5 49	6 09	6 71	7 35	8 01	8·69
2	9·39	10·11	10·85	11 61	12 39	13 19	14·01	14 85	15 71	16 58
3	17 47	18 37	19 29	20 22	21 17	22 13	23 11	24 10	25·11	26 13
4	27 16	28 21	29 27	30 35	31 44	32 55	33 67	34 80	35 94	37·09
5	38 25	39 42	40 60	41 80	43·01	44 23	45·46	46 71	47 97	49 24
6	50·52	51·81	53·11	54 42	55 74	57 07	58 41	59 76	61 12	62 44
7	63 87	65 26	66 66	68 07	69·49	70·92	72·36	73 81	75 27	76 64
8	78 22	79 71	81·21	82 72	84 24	85 77	87 31	88 86	90 42	91·98
9	93 55	95 13	96 72	98·32	99·93	101 55	103 18	104 82	106·47	108 12
10	109·78	—	—	—	—	—	—	—	—	—

NOTES.

1. The formula $D = \frac{2}{3} cbd\sqrt{2gd}$, devised by Mr. James B. Francis, was deduced by him from an elaborate series of experiments at Lowell, U S.A. (Lowell Hydraulic Experiments, New York, 1871) on a sharp-crested weir 10 feet long, of the full width of the stream, and with heads varying from 0 4 to 1 6 feet. Further experiments were made by Fteley, Stearns and others, from which Hamilton Smith has derived values for the coefficient *c* for such weirs varying in length from 2 to 19 feet [1]

For a 2-feet weir with a depth of 0·10 feet, the coefficient determined was 0·652 ; with a depth of 0·40 foot the coefficient attained its minimum, 0·636, and then increased gradually to 0·648 with a depth of 1 00 foot, the maximum depth of the experiments with this length of weir. The intermediate observations varied proportionally.

For a 19-foot weir with a depth of 0 10 foot, the coefficient was 0·657 ; with a depth of 0 70 foot the coefficient attained its minimum, 0·618, and then increased gradually to 0·623 with a depth of 1·60 feet, the maximum depth of the experiments with this length of weir. The intermediate observations varied proportionally.

[1] *Vide* Table No. 42, p. 220, of "Public Water Supplies", by Turneaure and Russell. Chapman and Hall, 1st edn., 1907.

2 Generally in tables for the discharge of gauging weirs constructed on Francis' formula the value of the coefficient c adopted is 0·512 throughout, although that given in Molesworth's Pocket Book is calculated with the value of c as 0 67 throughout. Judging from the above experiments, from the analogy of Bazin's coefficient (Appx. 9) and from general considerations, the coefficient must increase slightly as the depth of the discharge increases (see also the note to Appx. 10)

It is believed that, owing to the practical difficulty of gauging flows of great depth, experiments for determining their coefficients have never been made.

3. The Table given above has therefore been constructed in the following manner. The coefficients adopted were —

Depths in Feet	Coefficients	Depths in Feet	Coefficients	Depths in Feet	Coefficients	Depths in Feet	Coefficients.
0·00	0·600	2·50	0 623	5 00	0 635	7 50	0·646
0·50	0 605	3 00	0 625	5·50	0·638	8·00	0·647
1 00	0 610	3 50	0 628	6·00	0 640	8·50	0·649
1 50	0 615	4·00	0 630	6·50	0 642	9 00	0·650
2 00	0 620	4 50	0 633	7 00	0·644	9·50	0·650

The results thus obtained were afterwards slightly adjusted so as to get a series of discharges differing regularly from each other, and these were thereafter tabulated.

4 It must be remembered that the experiments quoted above were made on sharp-crested weirs, and therefore their results do not hold good for waste-weir walls as constructed in practice, which walls have crests of appreciable width.

Experiments [1] were made at the Cornell University Hydraulic Laboratory, U.S A., by Rafter and Williams, with weirs of different sections, 6·58 feet long and from 4 6 to 4 9 feet high, free access being provided for the air beneath the sheet of water Unfortunately the sections experimented with were not similar to those which are adopted in India for waste-weir walls. The following are the coefficients ascertained for weirs with vertical sides and horizontal crests, which are the ones nearest to the usual Indian sections.

COEFFICIENTS.

Width of Weir Crest in Feet.	Height of Still Water above Weir Crest in Feet.						
	2·0	2·5	3·0	3 5	4 0	4·5	5 0
2·62	0 53	0·54	0·57	0·59	0·62	0·65	0·69
6 56	—	0·46	0·47	0·47	0·48	0 49	0·50

The Table given in this Appendix will therefore be fairly exact for weirs having level crests with widths of from 3 feet to 4 feet and downstream batters of 1 in 4 (which will be the ones generally constructed in connection with storage reservoirs), and, especially, in the case of depths of water of from 5 feet to 6 feet, which will usually be the maximum flood depths on them.

5. The larger coefficients of discharge over a sharp-crested than over a broad-crested weir point to the desirability of forming a waste-weir wall with a sharp crest. This can be done by fixing an angle iron on the upstream side of the top of the wall and by bevelling that top downstream. A considerable increase of storage or shortening of the waste-weir can thus be effected at small expense.

[1] *Vide* Table No. 45, p. 224 of "Public Water Supplies", by Turneaure and Russell. Chapman and Hall, 1st edn., 1907.

APPENDIX 12.

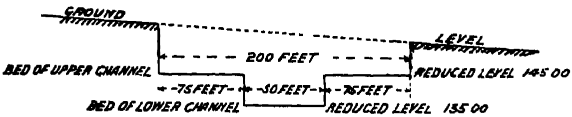
MÁLÁDEVI TANK PROJECT.

Temporary Waste-Weir Flood Calculations.—First Closure.

Table of Open Channel Discharges and Reservoir Flood-Absorption.

(*Vide* Chapter II., paragraph 137, page 109.)

Section of Open Channel.



Data.

The flood to commence when the channel is discharging at the rate of 5,300 cubic feet per second, or at the rate of about $\frac{1}{18}$ inch per hour run-off, which is a fair, small flood. To pass this discharge the total flood depth of the 50-foot channel will have to be 10 feet deep, so that the calculations will begin from Reduced Level 145·00. The rise of the reservoir surface level thereafter to be that due to a run-off at the rate of about 0·5 inch per hour from the catchment of 153 square miles.

In the first hour the reservoir will thus have to rise 4 feet; in the second hour, 3 feet, in the third hour, 2 feet; and, afterwards, 1 foot per hour, to deal with the assumed intensity of the flood.

The quantities discharged by the channel are calculated out foot by foot during the periods of rise.

Number of hours from Commencement of Flood.	At the end of each hour			During the hour.				
	Reduced Level of Reservoir Surface	Height of Reservoir Surface above Bed of Lower Channel	Height of Reservoir Surface above Bed of Upper Channel.	Total discharge of Lower Channel.	Total Discharge of Upper Channel	Increment of Reservoir Storage (or Flood Absorption)	Total Flood dealt with (Col 5 + Col. 6 + Col. 7)	Equivalent Run-off (of Col. 8) from Catchment.
1	2	3	4	5	6	7	8	9
		Feet.	Feet.	Million Cub Feet	Million Cub Feet	Million Cub. Feet.	Million Cub Feet	Inches.
1	146 00	11	1	5·205	0·161	29·086	157 046	0 44
	147 00	12	2	6 104	0 653	30·660		
	148 00	13	3	7 073	1 417	32 281		
	149·00	14	4	8 037	2 425	33·944		
2	150 00	15	5	11 950	4·929	35·643	172·787	0·49
	151 00	16	6	13·219	6 882	37·383		
	152·00	17	7	14 532	9·085	39·164		
3	153·00	18	8	23·838	17·646	40·994	173 136	0 49
	154 00	19	9	25 970	21·814	42·865		
4	155·00	20	10	56·336	52 375	44·771	153·482	0 43
5	156 00	21	11	60 750	62·459	46 746	169 955	0 48
6	157 00	22	12	65 250	73 008	48 792	187 050	0 53
7	158·00	23	13	69 250	84·877	50 881	205 008	0·58
8	159 00	24	14	74·250	96 447	53·015	223·712	0·63
9	160·00	25	15	78 750	107·552	55·192	241·494	0·68
		Totals	. .	520 523	541 730	621·417	1,683 670	4·75
Add total of Column 5			. .		520 523			
Total Discharge of Channels					1,062 253			
Percentage quantities					63·09	36 91	100 00	
Add total High-Flood Discharge of Channels at Reduced Level 160 00 for 15 hours						. . . .		8·19
		Total Run-off dealt with in 24 hours		.		.	.	12·94

The calculations thus show that with the open temporary waste-weir channel the Reservoir High-Flood Level will not rise above Reduced Level 160·00. At this level the discharge of the channel is 53,920 cubic feet per second, which is equal to a run-off of 0·546 inch per hour from the catchment of 153 square miles.

APPENDIX 13.

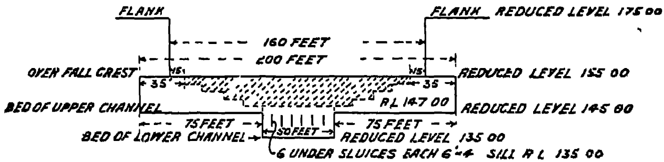
MÁLÁDEVI TANK PROJECT.

TEMPORARY WASTE-WEIR FLOOD CALCULATIONS—SECOND CLOSURE.

TABLE OF OUTLET HEADWALL DISCHARGES AND RESERVOIR FLOOD-ABSORPTION

(*Vide* Chapter II., paragraph 138, page 190.)

SECTION OF OUTLET HEADWALL.



Data.

The flood to commence when the crest is flowing 4 feet deep, *i.e.*, is discharging 5,706 cubic feet per second, or at the rate of about $\frac{1}{17}$ inch per hour run-off, which is a fair, small flood.

The reservoir surface will therefore be at Reduced Level 159 000 at the commencement. The rise of the reservoir surface level thereafter to be that due to a run-off at the rate of about 0 5 inch per hour from the catchment of 153·00 square miles

In the first and second hours, the Reservoir will thus have to rise 2 feet per hour, and, subsequently, 1 foot per hour, to deal with the assumed intensity of the flood.

The quantities discharged by the Headwall, &c., are calculated out foot by foot during the period of rise.

Note.

In construction the Headwall crest should be finished off with a central gap—as shown by dotted lines—so as to give it a greater discharging power, to deal with the flood earlier, and to lessen the action of the flood on the foundations.

Number of hours from commencement of Flood.	At the end of each hour.		During the hour				
	Reduced Level of Reservoir Surface.	Height of Reservoir Surface above Headwall Crest.	Total Discharge of Headwell Crest.	Total Discharge. of Under-sluices.	Increment of Reservoir Storage (or Flood-Absorption).	Total Flood dealt with (Col 4 + Col. 5 + Col 6).	Equivalent Run-off (of Column 7) from Catchment.
1	2	3	4	5	6	7	8
		Feet.	Million Cubic Feet.	Million Cubic Feet.	Million Cubic Feet.	Million Cubic Feet.	Inches
1	160 00	5	12·311	6 635	55·192	155·197	0·44
	161 00	6	17·011	6 635	57 413		
2	162·00	7	21 324	6 635	59 711	182·708	0·51
	163 00	8	26 417	6 635	61 986		
3	164·00	9	63 725	13·478	64 338	141·541	0·40
4	165 00	10	75 308	13·687	66 734	155 729	0 44
5	166 00	11	87 725	13 687	69·391	170·803	0 48
6	167 00	12	100 310	13·892	72 329	186·531	0·52
7	168 00	13	113 650	14 101	75·336	203·087	0·57
8	169 00	14	127 530	14·301	78 396	220 236	0 62
Totals			645 311	109·695	660·826	1,415·832	3 98
Add total of Column 4				645·311			
Total Outlet Headwall Discharge				755 006			
Percentage quantities				53·32	46·68	100 000	
Add total Outlet Headwall Discharge for 16 hours							6·71
Total Run-off dealt with in 24 hours							10·69

By calculation it is acertained that the discharge of the tail channels when flowing level with the crest of the Headwall (Reduced Level 155 00) will be about 80,000 cubic feet per second. As the maximum discharge to be dealt with is only 49,367 cubic feet per second (due to a run-off of 0 5 inch from the catchment), the Headwall will thus discharge throughout as a Clear Overfall Weir.

To determine the discharge of the Under-sluices, the approximate effective head on them is first calculated, *i e.*, the difference between the surface of the Reservoir upstream and that of the Tail Channel downstream. This latter is deduced from Appendix 10.

The calculations thus show that with the Outlet Headwall crest constructed as shown by the full line (much more so when built as shown by the dotted lines), the Reservoir High-Flood Level will not rise above Reduced Level 160·00. At this level the discharge of the Headwall and Under-sluices is 41,424 cubic feet per second, which is equal to a run-off of 0·42 inch per hour from the catchment of 153 square miles.

APPENDIX 14.

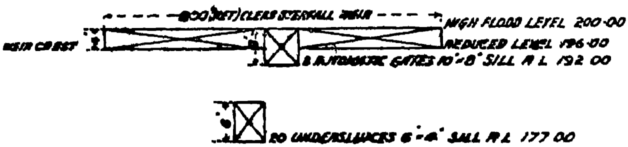
MÁLÁDEVI TANK PROJECT.

PERMANENT WASTE-WEIR FLOOD CALCULATIONS.

TABLE OF WASTE-WEIR DISCHARGES AND RESERVOIR FLOOD-ABSORPTION

(*Vide* Chapter III., paragraph 193, page 275.)

FLOOD SECTION OF WEIR.



Data.

The flood to commence when the reservoir level has risen 10 feet above the sills of the under-sluices, *i.e.*, to Reduced Level 187·00. Its initial amount will thus be 7,600 cubic feet per second, or at the rate of about $\frac{1}{13}$th inch per hour run-off, which is a fair, small flood.

All sluices (except the automatic gates) to be fully open and the weir crest unobstructed. The Reservoir surface to rise 1 foot per hour

These calculations do not take into account the discharge from the Outlet sluices, which will vary from 6,221 to 7,027 cubic feet per second. or from 0·06 to 0·07 inch per hour run-off, during the period considered,

Number of hours from Commencement of Flood	At the end of each hour.		During the hour				
	Reduced Level of Reservoir Surface	Height of Reservoir Surface above Bed of Waste-Weir Tail Channel	Total Discharge of Under-sluices	Total Discharge of Waste-Weir Wall Crest	Increment of Reservoir Storage (of Flood-Absorption).	Total Flood dealt with. (Col 4 + Col 5 + Col. 6)	Equivalent Run-off (or Column 7) from Catchment.
1	2	3	4	5	6	7	8
		Feet	Million Cubic Feet	Million Cubic Feet.	Million Cubic Feet	Million Cubic Feet.	Inches.
1	188 00	11	28 440	—	128·438	156 878	0·44
2	189·00	12	30 420	—	130 886	161 306	0 45
3	190 00	13	32 220	—	133·357	165 577	0·47
4	191 00	14	33 840	—	135 747	169 587	0·48
5	192 00	15	35 280	—	138·028	173·308	0·49
6	193 00	16	36 720	—	140·328	177·048	0 50
7	194 00	17	38 160	—	142·646	180 806	0 51
8	195 00	18	39 600	—	144·985	184·585	0 52
9	196 00	[1] 19	41 040	—	147·356	188 396	0·53
10	197 00	20	42 480	5 137	149·733	197 350	0 55
11	198 00	21	43 200	19·656	152·128	214 984	0 60
12	199 00	22	43 200	41 209	154·555	238 964	0 67
13	200 00	[2] 23	43 200	67·777	157·002	267 979	0·75
Totals			487 800	133 779	1,855·189	2,476 768	6·96
Add total of Column 4				487 800			
Total Waste-Weir Discharge . .				621 579			
Percentage quantities				25 10	74·90	100·00	
Add total Waste-Weir High-Flood Level Discharge for 11 hours (including automatic gates)							4·60
Total Run-off dealt with in 24 hours							11·56

[1] Masonry Crest of Waste-Weir Wall.
[2] At Reduced Level 200·00 the automatic gates come into action and the total Waste-Weir discharge becomes 41,284 cubic feet per second, or, at the rate of 0 418 inch per hour run-off from the catchment of 153 square miles

To determine the discharge of the Under-sluices, the approximate effective head on them is first calculated, *i e*, the difference between the surface of the Reservoir upstream and that of the Tail Channel downstream This latter is deduced from Appendix 10.

At first the sluices have partly drowned and partly clear openings, and finally, wholly drowned openings.

APPENDIX 15.

RECAPITULATION OF MÁLÁDEVI TANK PROJECT.

(*Vide* Chapter V., paragraph 244, page 346.)

1	2	3
Items.	Amount. Rs	Total. Rs.
I. WORKS		
1 *Headworks*		
Preliminary expenses—survey, gauging, &c	Previous expenditure to be written off	
2. *Land*		
Compensation for acquisition		75,000
3. *Masonry Works.*		
Waste-Weir	1,56,007	
Outlet	1,22,372	2,78,379
4. *Buildings.*		
Bungalow and outhouses	6,727	
Overseer's quarters	1,121	
Watchmen's quarters	1,052	
Temporary buildings	1,100	10,000
5. *Earthwork.*		
Dam embankment, pitching, puddle trench, &c		8,46,954
6 *Plantation*		
Planting and preserving trees around the Reservoir		5,000
7 *Miscellaneous*		
Maintenance during construction	10,000	
Boat and boat-house	1,000	11,000
Total I. Works		**12,26,333**
II. ESTABLISHMENT		
Construction Establishment (at 18 per cent. on the cost of the project exclusive of land charges—i e, on Rs 11,51,333)	2,07,240	
Direction and accounts (at 5 per cent on ditto, ditto)	57,560	
Total II. Establishment		**2,64,800**
III. TOOLS AND PLANT		
Tools and Plant (at 5[1] per cent on I WORKS—less land charges) say :—		**58,000**
A **Total Direct Charges**		**15,49,133**
IV. SUSPENSE (NOT AFFECTING CHARGES TO GRANT)		
Capitalisation of abatement of Land Revenue	75,000	
Leave and pension allowances (at 14 per cent. on II. ESTABLISHMENT)	37,100	
Interest on I. WORKS (at 2 per cent on year's expenditure and 4 per cent. on previous expenditure) [2]	1,22,600	
B **Total Indirect Charges**		**2,34,700**
Grand Total, Direct and Indirect Charges		**17,83,833**

[1] An extra allowance beyond the usual one of 1½ per cent has been made to provide for the large amount of plant necessary.

[2] The work is estimated to take 5 years for completion, hence the interest charges amount to 2 per cent per annum for 5 years on the whole sum at charge.

APPENDIX 16.

ABSTRACT ESTIMATE OF MÁLÁDEVI TANK PROJECT.

(*Vide* Chapter V., paragraph 244, page 346)

1	2	3	4	5	6	7
Quantity	Unit.	Items	Rate.	Per.	Amount.	Remarks.
			Rs A. P		Rs.	
		I DAM EMBANKMENT				
		1 *Puddle Trench*				
1,140,841	Cft	Excavation in soil [1] .	1 8 0	100	17,113	
182,495	,,	Ditto in muram [1] . .	1 8 0	,,	2,737	
23,803	,,	Ditto in rock [1] . .	4 0 0	,,	952	
27,495	,,	Ditto rock in puddle trench drain [1] . .	6 0 0	,,	1,650	
106,675	,,	Ditto nulla puddle trench [1]	0 12 0	,,	800	
1,347,139	,,	Filling dam puddle trench	1 4 0	,,	16,839	
213,350	,,	Ditto nulla ,, ,,	1 0 0	,,	2,134	
4,219	Rft	Puddle trench drain, excavation and filling .	1 0 0	Rft.	4,219	
			Total	*Rs*	46,444	
		Add contingencies at 5 per		*cent*	2,322	
			Grand total	*Rs*	48,766	
		2. *Central Wall*				
18,361	Cft.	Excavation in rock [1] .	3 0 0	100	551	
50,386	,,	Concrete . :	15 0 0	,,	7,558	
17,358	,,	Masonry facing . .	32 0 0	,,	5,555	
			Total	*Rs*	13,664	
		Add contingencies at 5 per		*cent*	683	
			Grand Total	*Rs*	14,347	
		3. *Drainage Works.*				
		(*a*) *Foundation, Surface, and Cross Drains.*				
610,012	Cft.	Excavation in soil [1] .	0 8 0	100	3,050	
235,860	,,	Filling with clay . .	0 12 0	,,	1,769	
188,280	,,	Ditto gravel . .	2 0 0	,,	3,766	
			Total (*a*)	*Rs.*	8,585	

[1] In all estimates the rates for excavation are reduced by the value of the material to constructional items (dam, drystone toe, &c.), where full rates are allowed.

ABSTRACT ESTIMATE—*continued*

1	2	3	4	5	6	7
Quantity	Unit	Items.	Rate.	Per.	Amount.	Remarks
			Rs A. P.		Rs.	
		I DAM EMBANKMENT—*continued.*				
		(*b*) *Downstream Drain.*				
492,844	Cft	Excavation in soil .	0 12 0	100	3,896	
492,844	,,	Filling . .	2 0 0	,,	9,857	
210	Rft	Rear drain below dam .	1 0 0	Rft	210	
			Total (b)	*Rs*	13,963	
			Total (a)+(b)	*Rs*	22,548	
		Add contingencies at 5 per		*cent*	1,127	
			Grand Total	*Rs*	23,675	
		4 *Drystone Toe*				
51,331	Cft	Excavation in rock [1]	3 0 0	100	1,540	
68,533	,,	Excavation in soil [1] .	0 4 0	,,	171	
553,383	,,	Concrete . .	15 0 0	,,	83,007	
4,161,473	,,	Drystone . . .	4 0 0	,,	1,66,459	
			Total	*Rs.*	2,51,177	
		Add contingencies at 5 per		*cent*	12,559	
			Grand Total	*Rs*	2,63,735	
		5 *Dam Embankment*				
26,836,904	Cft	Embankment . .	1 8 0	100	4,02,553	
34,822	Sft	Pitching 12″ thick	6 0 0	,,	2,089	
187,081	,,	Ditto 12″ to 18″ thick .	8 0 0	,,	14,966	
191,855	,,	Ditto 18″ thick . .	10 0 0	,,	19,186	
20,935	,,	Ditto 24″ thick . .	15 0 0	,,	3,140	
			Total	*Rs*	4,41,934	
		Add contingencies at 5 per		*cent*	22,097	
			Grand Total	*Rs*	4,64,031	
		6. *Masonry Wall at Top.*				
76,657	Cft.	Coursed rubble masonry	22 0 0	100	16,865	
36,516	,,	Concrete .	16 0 0	,,	5,843	
4,296	Rft.	Coping, extra . .	0 8 0	Rft.	2,148	
			Total	*Rs.*	24,856	
		Add contingencies at 5 per		*cent*	1,243	
			Grand Total	*Rs.*	26,099	
		7. *Miscellaneous.*				
		Road to bungalow from Rambhori . .	Lump	—	2,100	
		Works roads . .	Ditto	—	2,100	
		Sundries . . .	Ditto	—	2,100	
			Grand Total	*Rs*	6,300	
Grand Total—I. Dam Embankment.				*Rs.*	**8,46,954**	

[1] See footnote on p. 377

ABSTRACT ESTIMATE—*continued.*

1	2	3	4			5	6	7
Quantity.	Unit.	Items.	Rate.			Per.	Amount.	Remarks
		II. WASTE-WEIR	Rs.	A	P.		Rs.	
		A. Clear Overfall Section						
71,197	Cft	Excavation in soil and muram [1] . . .	0	12	0	100	534	
37,895	,,	Concrete . . .	15	0	0	,,	5,684	
22,729	,,	Masonry facing . .	32	0	0	,,	7,273	
3,275	,,	Block-in-course masonry	50	0	0	,,	1,638	
1,493	Rft	Cornice . . .	1	0	0	Rft	1,493	
		Total				*Rs*	16,622	
		Add contingencies at 5 per				*cent*	831	
		Grand Total				*Rs*	17,453	
		B Under-Sluice Section						
42,136	Cft.	Excavation in muram [1] .	0	8	0	100	211	
37,929	,,	Ditto in soft rock [1] .	1	8	0	,,	569	
32,780	,,	Ditto in hard rock [1] .	4	0	0	,,	1,311	
57,188	,,	Concrete hearting . .	15	0	0	,,	8,578	
17,192	,,	Masonry facing . .	32	0	0	,,	5,501	
2,205	,,	Block-in-course masonry	50	0	0	,,	1,102	
16,348	,,	Ashlar masonry . .	60	0	0	,,	9,809	
2,821	,,	Ditto arching . .	70	0	0	,,	1,975	
263	,,	Corbels . . .	60	0	0	,,	158	
80	No.	Sluice sills and lintels .	8	0	0	Each	640	
562	Rft	Cornice . .	1	0	0	Rft	562	
522	Cwt.	Iron rails . .	4	0	0	Cwt	2,088	
130	Cft.	Cut teak woodwork .	4	0	0	Cft.	520	
20	No.	Sluice gates, 6′ × 4′ .	1,200	0	0	Each	24,000	
		Total				*Rs*	57,024	
		Add contingencies at 5 per				*cent.*	2,851	
		Grand Total				*Rs*	59,875	
		C. Automatic Gate Section						
18,736	Cft	Excavation in muram [1] .	0	8	0	100	94	
5,350	,,	Ditto in soft rock [1] .	1	8	0	,,	80	
5,787	,,	Ditto in hard rock [1] .	4	0	0	,,	231	
15,367	,,	Concrete . . .	15	0	0	,,	2,305	
5,925	,,	Masonry facing . .	32	0	0	,,	1,896	
3,949	,,	Block-in-course masonry	50	0	0	,,	1,960	
706	,,	Arching . . .	55	0	0	,,	388	
144	Rft.	Cornice . .	1	0	0	Rft	144	
8	No.	Automatic gates, 10′ × 8′	1,800	0	0	Each.	14,400	
		Total				*Rs*	21,498	
		Add contingencies at 5 per				*cent.*	1,075	
		Grand Total				*Rs.*	22,573	

[1] See footnote on p. 377

ABSTRACT ESTIMATE—*continued.*

1	2	3	4	5	6	7
Quantity	Unit.	Items	Rate.	Per.	Amount	Remarks.
			Rs. A. P.		Rs.	
		II WASTE-WEIR—*contd.*				
		D. Arcade Footpath.				
3,269	Cft.	Concrete . . .	15 0 0	100	490	
17,179	,,	Block-in-course masonry	32 0 0	,,	5,497	
5,223	,,	Arching . . .	55 0 0	,,	2,873	
1,020	,,	Corbels . . .	60 0 0	,,	612	
160	No	Cut- and ease-water caps	5 0 0	Each	800	
1,035	Cft.	Cut teak woodwork .	4 0 0	Cft	4,140	
2	No.	Travelling winches .	250 0 0	Each	500	
		Tramway . . .	Lump	—	1,500	
			Total	*Rs*	16,412	
		Add contingencies at 5 per		*cent*	821	
			Grand Total	*Rs.*	17,233	
		E Protective Works				
82,094	Cft.	Excavation in soil [1] .	0 8 0	100	410	
17,671	,,	Ditto in soft rock [1] .	1 8 0	,,	265	
12,162	,,	Ditto in hard rock [1] .	5 0 0	,,	608	
38,372	,,	Concrete . . .	15 0 0	,,	5,757	
20,522	,,	Coursed rubble masonry, 1st sort . . .	32 0 0	,,	6,567	
27,913	,,	Ditto, 2nd sort . .	24 0 0	,,	6,699	
		Protective bank near Rambhori . .	Lump	—	800	
			Total	*Rs*	21,106	
		Add contingencies at 5 per		*cent*	1,055	
			Grand Total	*Rs*	22,161	
		F Approach and Tail Channels				
3,916,853	Cft	Excavation in soil [1] .	0 6 0	100	14,688	
327,600	,,	Embankment . .	0 6 0	,,	1,228	
			Total	*Rs.*	15,916	
		Add contingencies at 5 per		*cent*	796	
			Grand Total	*Rs*	16,712	
	Grand Total—II. Waste-Weir	. .		*Rs.*	**1,56,007**	

[1] See footnote on p. 377.

ABSTRACT ESTIMATE—*concluded*

1	2	3	4			5	6	7
Quantity	Unit	Items	Rate.			Per.	Amount	Remarks.
			Rs.	A	P.		Rs.	
		III. OUTLET [1]						
297,737	Cft	Earth excavation [2]	0	6	0	100	1,116	
31,703	,,	Rock excavation [2]	2	0	0	,,	634	
106,800	,,	Puddle filling	1	8	0	,,	1,602	
292,562	,,	Concrete	15	0	0	,,	43,884	
87,683	,,	Masonry facing	32	0	0	,,	28,059	
2,569	,,	Block-in-course masonry	50	0	0	,,	1,285	
3,193	,,	Arch work	55	0	0	,,	1,756	
2,235	,,	Coping	50	0	0	,,	1,117	
387	,,	Cornice	60	0	0	,,	232	
84,303	,,	Drystone toe	4	0	0	,,	3,372	
6	No.	Sluice gates, 6′ × 4′	2,000	0	0	Each	12,000	
2,182,500	Cft	Channel excavation [2]	0	6	0	100	8,184	
569,637	,,	Dam embankment	1	8	0	,,	8,545	
10,968	Sft	Pitching	6	0	0	,,	658	
1,360	Cft.	Masonry of crest wall	24	0	0	,,	326	
2	No	Turbine sluices	700	0	0	Each	1,400	
380	Cwt	Turbine pipes	6	4	0	Cwt	2,375	
					Total	*Rs*	1,16,545	
		Add contingencies at 5 per				*cent*	5,927	
		Grand Total—III. Outlet				*Rs.*	**1,22,372**	
		IV BUILDINGS.						
		Bungalow	—				4,893	
		Out-houses	—				1,834	
		Overseer's Quarters	—				1,121	
		Watchmen's Quarters	—				1,052	
		Temporary Buildings	—				1,100	
		Grand Total—IV. Buildings	—			*Rs*	**10,000**	

[1] This estimate provides for all work in the 200 feet length occupied by the temporary Waste-Weir Channel.

[2] See footnote on p. 377

APPENDIX 17.

MÁLÁDEVI TANK PROJECT.

Table of Reservoir Contents at Each Contour.

(*Vide* Chapter I., paragraph 21, page 33.)

1	2	3	4	5	6
Reduced level of Contour	Square root of area of Contour	Area of Contour.	Contents between successive Contours	Contents up to each Contour.	Remarks.
		Square feet	Cubic feet	Cubic feet.	
135	**4,328**	**18,731,584**	—	—	**Outlet sill.**
136	4,427	19,598,320	19,163,320	19,163,320	
137	4,527	20,493,729	20,044,359	39,207,679	
138	4,626	21,399,876	20,945,169	60,152,848	
139	6,725	22,325,625	21,861,117	82,013,965	
140	4,825	23,280,625	22,801,458	104,815,423	
141	4,924	24,245,776	23,761,567	128,576,990	
142	5,023	25,230,529	24,736,519	153,313,509	
143	5,122	26,234,884	25,731,073	179,044,582	
144	5,222	27,260,284	26,747,417	205,791,999	
145	**5,321**	**28,313,041**	**27,786,529**	**233,578,528**	
146	5,465	29,866,225	29,086,177	262,664,705	
147	5,609	31,460,881	30,660,097	293,324,802	
148	5,754	33,108,516	32,281,194	325,605,996	
149	5,898	34,786,404	33,944,004	359,550,000	
150	6,042	36,505,764	35,642,628	395,192,628	
151	6,186	38,266,596	37,382,724	432,575,352	
152	6,330	40,068,900	39,164,292	471,739,644	
153	6,475	41,925,625	40,993,758	512,733,402	
154	6,619	43,811,161	42,864,937	555,598,339	
155	**6,763**	**45,738,169**	**44,771,209**	**600,369,548**	
*	*	*	*	*	
177	10,203	104,101,209	103,013,304	2,202,486,148	Sill of waste-weir under-sluices.
*	*	*	*	*	
196	12,188	148,547,344	147,356,121	4,498,935,205	Masonry crest of waste-weir wall.
*	*	*	*	*	
200	12,579	158,231,241	157,001,700	5,112,353,401	H F L. and F S L. Crest of teak shutters

Notes

1. Surveyed contours, &c., are shown in heavy type figures.
2. Interpolated contours, &c., are shown in light type figures.
3. The table is constructed on the assumption that the square roots of the areas of the interpolated contours between those of the surveyed ones are in arithmetical progression.

APPENDIX 17^B.

THE EARLY HISTORY OF THE WÁGHÁD DAM.

(*Vide* Chapter II., paragraph 147, page 199.)

THE early history of the Wághád Dam in the Násik District of the Bombay Presidency is instructive on account of the accidents which befell the work, for failures teach more than successes.

The centre line of this dam on the right bank runs on an undulating elevated ridge with one pronounced depression, or saddle: it then descends with a steep slope to the bed of the small Wághád River and rises therefrom precipitously on the left bank, on which bank is the tunnel outlet, thus situated in as bad a position as possible (p. 282). Further on, and separated by high ground from the dam, is the 650 feet long waste-weir having a tail channel with excessive slope (p. 246). The country is in the Deccán trap formation and the surface of the ridge is generally of *muram* with fissured rock below: at river bed level sound rock is soon reached. The catchment area is only 29 square miles, but as it extends to the crest of the Western Ghárts about 5 miles from the dam, is subject to heavy rainfall. The maximum height of the dam was designed as 95 feet, *plus* 3 feet allowed for settlement, or considerably higher than any previously constructed by the British Government. Its length is 4,160 feet, and it was formed of pure black cotton-soil, instead of a mixture with it of gritty material to prevent it from slipping (p. 156).

In 1881 construction was started by constructing the low right flank embankment: this no doubt was done to collect labour gradually, but had the unfortunate effect of closing the saddle there, which might otherwise have been used as a temporary waste-weir when the gorge embankment was under construction (p. 185).

In 1882 the gorge itself was attacked by a contractor, but progress was so slow that there was no chance of completing its dam before the monsoon. When the base had been raised some 20 feet, it was decided to protect it from being washed away by monsoon floods by means of a downstream curtain, or retaining wall: unfortunately, the section of that was too light, the wall was breached early and the earthen dam then rapidly followed.

In 1883 the gorge embankment was recommenced departmentally, but again, unfortunately, the work was not pushed from the start, as it should have been, and very little progress was made at first with the building of the outlet tunnel, which ought to have been constructed before hand The main dam on its right flank had thus to be raised independently of it, and after the culvert had been completed, had to be carried across the resulting hollow, thus forming a very weak junction in the earthwork where that was very high. In April cholera suddenly broke out, and nearly all the scared work-people fled to their neighbouring villages Eventually, they were brought back under an application of the sections of the Canal Act dealing with forced labour in great emergencies, which was all the milder as it was the first time effect had been given to these sections. Unfortunately an invaluable month was thus lost, and as the top of the earthwork, in the middle of May, was more than 20 feet below waste-weir crest level, a crest bank of slight section was raised on the upstream side of the main dam, leaving the rest of its top as a terrace, while a 50-foot cut some 10 feet deep was made through the saddle of the waste-weir. Work was thereafter carried on with feverish haste : the earth was brought on to the reduced section from both flanks, but the carts conveying it neither crossed each other nor turned on it, being deflected midway for the return journey down the downstream slope of the crest bank to the lower terrace. At the downstream edge of this strong wooden barriers and shouting human guards prevented the terrified oxen from rushing down the remaining 60 feet of slope to the river bed below. Thus the 300-foot long crest dam was raised daily by about a foot vertical, but there was not time, nor opportunity for pitching its upstream slope. Heavy rains fell at the end of June, and the reservoir, previously but a large puddle, rose rapidly and became a sheet of water over 600 acres in extent. The waves washed away the toe of the green earthwork of the crest bank, but spent most of their force on the foreshore they themselves had produced, and thus it was practicable to protect the rest of the crest dam from erosion by facing it with bamboo matting torn from work sheds and pinned down at top and bottom by lines of pitching stones. This protection lasted for some hours, during which time the cut in the waste-weir was able to lower the flood level.

The end, however, came soon, for during the night of 7th July, 1883, some 5 inches of rain fell at the dam site and over 10 inches on the Ghát line, the reservoir rose nearly to the top of the dam and the spray of the waves was carried over that. All that could be done by the night

watchmen, at the peril of their lives, was to raise the left flank of the crest dam a few inches so as to cause the breach to take place on the right flank (compare Fig 28, p. 244), where it would be least disastrous: in this they were successful At 9 a m. on 8th July the dam began to breach, and in six hours the right half of the gorge embankment had been swept away by the 625 million cubic feet of water thus released. The whole of the narrow valley below was raised for some distance by *detritus*, about 7 feet deep, in which were embedded great boulders torn from the bed and side of the gorge, one of which was estimated at 30 tons in weight · the course of this was arrested some 200 feet downstream, by a large tree which was uprooted by the impact At the same time pieces of the earth of the dam were rolled down as solid masses, and crystals in cavities of the rock banks were uninjured. For several miles downstream the narrow and tortuous river channel lies in a forest-clad, hilly area, so that the loss of life due to the breach was confined to—one donkey! Half of the dam was left standing as a nearly vertical cliff 85 feet high, and the base of this was protected by a drystone bank from erosion by the river, which thereafter dwindled to a small stream. The actual high-flood discharge of the river was estimated as considerably exceeding the designed provision of 18,000 cusecs, *i.e*, at the rate of 1 inch an hour run-off from the catchment.

At the end of the 1883 monsoon the third and last attempt was successfully made to close the gorge by an earthen dam: the decision to do this was influenced by considerations of economy, for, from the engineering point of view, a masonry dam should have been substituted for it (top of p. 77). This time rapid progress was made from the start, and the dam was raised to 8 feet below its designed top by 27th April, 1884 On the morning of that day a hair crack was seen near the defective junction at the outlet this gradually extended up the downstream slope of the dam to near the top, where it formed a large loop and descended towards the left bank back to the ground Nothing was heard during that night, but next morning the dam was found to have subsided to a shapeless mass (Plate 15^B). There were two nearly vertical drops of 10 feet (near the top) and 15 feet (lower down), parallel to the centre line (Fig. 17, p. 199), while at right angles to it at the flanks were precipices of much greater height. The surfaces of all these cliffs were laminated by the great pressure of the movement, and striated by the grit in the subsiding earth; the downstream toe was pushed forward from 50 to 80 feet, and its layers were tilted upwards from the centre line to about 20° (p 198). Steps were at once taken to dress the cliffs to regular

slopes and to slope and terrace the rest of the downstream slope so as to shed the rainfall off as uniformly as possible. A drystone toe wall (Fig. 3, p 113) and a berm were formed at the downstream end of the subsidence to prevent further motion, and a safety crest wall of drystone with an earthen core was raised to some 15 feet in height on the top of the upstream slope of the dam which had not suffered any displacement. The cut in the waste-weir was deepened 2 feet and increased to 100 feet in width, so as earlier to tap floods and to discharge them more effectually The saddle in the long right flank embankment was cut down to form a safety escape (p. 218) with bed about 25 feet below dam top for a length of about 300 feet: this was done in some three weeks by 1,700 labourers who simply covered the working area The whole of these works were completed before the monsoon, and that season passed uneventfully. all that the strong maintenance gang had to do was continually to fill in the cracks and make up the minor subsidences as they occurred (p 339).

In the following season three long and deep cross drains were excavated in the gorge embankment to bed level, normally to the centre line, in timbered trenches and filled with drystone, to drain, and thus consolidate, the subsided earthwork pierced by them (p. 201). The toe works were strengthened by a large earthen berm (p 202) The safety cut in the right flank embankment was closed; and the upper part of the dam was gradually raised to its designed height, the safety crest dam being removed and replaced by the ordinary section In 1886 the work was completed, and for many years subsequently did not give much trouble. Unfortunately before the 1919 monsoon a very serious slip took placed on the upstream side of the outlet and had hastily to be repaired This subsidence still (1926) continues and has to be made good. It is unusual in that it occurred on the upstream side, but it is another instance of damage taking very long to develop at a defective place (p 165). It is probably due to the way, described above, in which the dam was raised over the outlet culvert The remedy suggested is that the whole of the slip should be cut out, carefully remade with gritty soil and supported by a strong earthen berm or retaining wall. The 100-foot safety cut in the waste-weir has been closed by a headwall in which are fixed twelve 10 by 8 feet Reinhold automatic gates (p. 268), which discharge into a large water cushion, and these have proved most successful. Owing to the steep slope downstream this cut scoured a deep channel for itself and carried down many large slabs of fissured rock, some of considerable size (p. 245).

APPENDIX 18.

SPECIFICATIONS FOR THE PUDDLE TRENCH, DAM EMBANKMENT AND PITCHING.

(*Vide* Chapter V, paragraph 245, page 346.)

INDEX TO CLAUSES.

SPECIFICATION FOR EARTHEN DAMS

1. General.—The dam is to consist of an earthen embankment with puddle and concrete trenches, as shown on the drawings. It is to be faced with pitching on the water-slope and to be provided with an outlet (s) and waste-weir (s). All works in connection with it shall be executed in strict accordance with the plans, estimates, and these specifications except when written orders by the Engineer permit deviations from them to be made. Only the best materials and the soundest form of construction shall be permitted, and every thing shall be executed in a thoroughly workmanlike way.

The following specifications proceed in the order in which the works will be carried out :—

I. THE PUDDLE TRENCH.

2. Position and Extent.—The puddle trench shall be excavated continuously along the centre line of the dam and shall extend from the high-flood margin on one bank to that on the other bank.

3. Depth and Formation.—The trench shall be excavated until sound rock, or impervious soil, is met with, and shall be carried into the former for a minimum depth of 1 foot. Where sound rock is not met with but compact subsoils exist, it may be stated as a general guide, the depth of the trench shall not be less than half the full supply pressure head of the reservoir at the point considered, and, where the subsoils are less compact, the depth shall not be less than that pressure head. In both these cases the bottom of the puddle trench shall be carried for at least one-tenth of that pressure head into sound, retentive, clayey material which shall extend well below it.

The puddle trench shall not be continued below the foundation of the outlet culvert, but, if the latter is crossed at a lower level, shall there be replaced by a concrete trench well keyed into the puddle at its flanks and supporting the culvert.

The puddle trench shall be replaced by a concrete trench when the pressure head is great and the sub-strata are of fissured, hard material, as the Engineer by written order may direct. The concrete shall be laid and consolidated as specified in clause 6.

4. Side-slopes.—The ratio of the side-slopes shall be determined by the Engineer from a consideration of the nature of the soils through which the trench passes, but in no case shall it be less than 1 in 8 in rock, nor less than 1 in 4 in soils, nor greater than is necessary to ensure the stability of the sides of the trench until it is filled in, and the sufficiency of the thickness of the filling When the side-slopes vary, they shall be steepest at the base and flattest at ground level, and nowhere shall the sides be taken out vertically. Shoring shall not be resorted to if it can possibly be avoided, but, where it is necessary, it shall be made in short lengths with slopes battering not less than 1 in 8, and the trench shall be rapidly filled in

5. Bottom-width.—The bottom width of the trench shall not be less than one-eighth of the full-supply pressure head *plus* 3 feet, nor less than 6 feet. It shall be taken out level in cross-section, except where a key trench shall be ordered

(clause 6), and shall not have any vertical steps in longitudinal section, all changes of level being effected by gentle slopes.

6. Key Trench.—If there is fissured rock at the bed of the puddle trench, a key trench not less than 3 feet wide, shall be excavated along the centre line of the dam, and shall extend both vertically and horizontally, at least 1 foot into sound rock. It shall be filled with fine rich concrete in well-rammed 4-inch layers constructed all at once, one on top of the other, until the whole key trench is completed, and on this filling a concrete wall shall be carried up at least 2 feet above the bed of the main trench, and shall be finished off with battering sides and a smooth, rounded top

7. River Crossing—Central Wall.—At the river crossing the puddle trench shall be replaced by a central wall of which the foundations shall be carried at least 2 feet into perfectly sound, unfissured rock, and the superstructure shall be raised to a height not less than one-eighth of the full-supply pressure head. The superstructure shall have a hearting of concrete, and its sides faced with masonry, and these shall batter not less than 1 in 8; its top shall not be less than 6 feet wide and shall be finished off with a longitudinal key recess to secure proper union with the embankment. The central wall shall be continued well into the natural banks of the river, and shall be keyed well into the puddle trench, which shall be thickened out to overlap it on both sides for a length not less than one-quarter of the full-supply pressure head. The central wall shall be constructed with smooth sides, plastered upstream with mortar or luted with clay, and small vertical pilasters and recesses shall be formed on its upstream side so as to make it key thoroughly with the earthwork of the dam On the downstream side of this wall shall be constructed a drain leading to the rear drain (clause 17).

8. Minor Drainage Crossings.—At these the puddle trench shall be deepened and widened, or shall have a key trench constructed at its base, or shall be replaced by a central concrete wall, as the Engineer shall direct in writing after taking into consideration the depth of the full-supply pressure head and the nature of the subsoils.

9. Springs in the Trench.—All large springs in the trench

shall be led in pipes to its downstream side, and shall be carried up with the trench filling. When the trench filling has reached a sufficient height, the pipes shall be carefully plugged with fine cement mortar Small quantities of water shall be carefully drained off so as not to affect the filling of the trench in any way

10. Material for Filling.—The material to be employed for filling shall be the most retentive clay procurable within one-half a mile of the site it shall be quite free from vegetable or slushy matter and earth impregnated with salt, and shall not be liable to deterioration by the infiltration of water. It shall not be mixed with gritty soils or with sand, and shall be only slightly damp

11. Filling of Trench.—The trench shall not be filled until it has been passed by the Engineer himself, and he shall carefully consider, by taking into account the nature of the subsoils met with and the full-supply pressure head of the reservoir, whether the depth is sufficient to ensure the staunchness of the work.

Just before filling is commenced, the sides and bottom of the trench shall be slightly roughened, so that the new material may unite perfectly with the natural soil. Where the bed of the trench is in rock or other hard material, it shall be swept and washed free from dust, etc. The bed thus prepared, shall be wetted and the first layer of the filling material, which shall have been made into stiff plastic balls, shall be thrown on to the bed and trodden so as to secure a perfect junction with it. The rest of the filling shall be constructed in layers not exceeding 3 inches in thickness, and these shall be smoothened and then thoroughly consolidated by ramming or by rolling with heavy rollers, so as completely to fill the trench from side to side, after its completion, each layer shall be wetted slightly to receive and unite with the next layer The material of the filling shall be only slightly damp, and all clods in it shall be broken up. The surface of the layers shall rise downstream at an inclination of about 1 in 6.

To cut off any leakage planes there may be, at vertical and horizontal intervals and breaking joint with each other shall be excavated wedge-shaped dowel trenches, 3 feet wide

at top and 1 foot deep, and these shall be refilled with the clayey material and thoroughly rammed before the upper layers are constructed.

Where sand pockets occur at the upstream side of the trench, they shall be excavated out, as far as practicable, just before the filling reaches them, and the filling shall be thoroughly worked into the cavities thus formed. If much sand is met with, the trench shall be widened on the upstream side to such extent as the Engineer may direct.

The filling shall be continued until the trench is completely filled, and up to 2 feet above ground surface; this upper part shall be constructed together with the embankment on each side, and thereafter the embankment shall be carried uniformly over it. If the construction of the embankment is not at once to be undertaken, the puddle shall be covered over with 2 feet of soil to prevent it from cracking—for the same reason the layers of the puddle trench filling shall be constructed, one on the top of the other, as rapidly as possible Should, however, cracks appear, the defective layers shall at once be cut out and properly remade.

12. Puddle Trench Drain.—When such a drain has been designed, it shall be carefully constructed of the specified dimensions in a trench excavated in the main bed of the puddle trench and founded throughout on an unyielding foundation, and shall have a continuous longitudinal slope to its outfall, which shall be continued across all depressions in the puddle trench on a masonry or concrete substructure. The side walls and covering slabs shall be constructed of as long stones as possible, laid in mortar, with half-inch dry joints at intervals of 5 feet intermediate with each other at the top and upstream side. The whole shall be surrounded with dry material (in accordance with the detailed drawing), and this shall be carefully covered with the material of the trench filling for a depth not exceeding 3 feet. After the whole drain has thus been completed, it shall be tested with water to see that it is perfectly free from obstructions, and, when this has been proved to be the case, the filling of the main trench shall be resumed and completed Should the flow be obstructed, the defective part shall be cut out and properly remade.

Wherever the levels of the ground permit, the drain shall be led out of the dam and allowed to discharge into the natural drainage channels, and shall in this case be constructed in sections independent of each other, each starting at least 5 feet downstream of the end of the upper section.

II. The Foundation of the Dam.

13. Clearance of Site.—The whole site to be covered by the embankment shall be cleared for its full width of all rubbish, loose stones, surface fissured rock (on the upstream side), powdery, greasy or cracked earth, silt, sand, and all soils charged with lime or other salts, or in other respects unsuitable for the foundation All trees, shrubs and vegetation growing on the site shall be completely rooted out, and the holes thus formed shall have their sides neatly sloped and filled with rammed clayey earth. For small dams the base shall be harrowed to secure watertight connection with the embankment.

Particular care shall be taken thoroughly to clear the site where the dam crosses the main and tributary streams, water-courses, &c., so as to obtain a perfectly sound foundation at these places.

The material thus removed shall not be utilised for the construction of the dam without the written permission of the Engineer, and, if not utilised, shall be neatly spread or disposed of as may be directed.

The foundation of the dam shall consist of sound material not liable to deteriorate under the action of water, nor to slip, and shall be able to support the dam while sustaining only a small and uniform amount of compression.

14. Foundation Benches and Trenches.—The whole site outside the puddle trench shall be excavated into foundation benches parallel to the centre line of the dam, with a width from crest to crest of 20 feet and a depth of trough of 3 feet, unless otherwise directed in writing or shown on the drawings. The side of each bench next the centre line of the dam shall be excavated with a slope about parallel to that of the dam on that side, and the slope of the other side shall be adjusted accordingly.

At the base of the foundation benches shall be excavated a series of trenches with sides sloping at 1 in 4, 3 feet wide at bottom and 3 feet deep, and with a central trench on each side of the main puddle trench 4 feet wide at bottom and 5 feet deep. On the upstream side these shall be filled with the most retentive clay procurable within half a mile of the site, and this shall be thoroughly well rammed On the downstream side the trenches shall be filled with quarry spauls, small rubble, &c , covered with fine dry material, to form drains, which shall have a continuous fall throughout their course, and shall be led, at intervals of about 300 feet, out of the dam by cross drains of similar section and construction into the downstream drain (clause 16) or to natural outfalls. The lower drains shall not be begun nearer than 5 feet from the downstream ends of the cross drains of the upper ones.

Where the dam rests on steep end-long ground, the foundation benches shall be excavated at right angles to the axis of the embankment, and with their main slope rising towards the surface of the ground downstream On the upstream side of the puddle trench the benches shall be divided into sections not more than 50 feet long, each separated from the others by unexcavated strips 5 feet wide, running parallel to the centre line of the dam, and, on the reservoir side of these strips shall be stop trenches 3 feet wide at bottom and 3 feet deep, filled with carefully rammed clay On the downstream side of the puddle trench the foundation trenches shall be excavated at right angles to the dam in the furrows of the benches, and shall be led straight out of the dam, independently of each other.

15. The Surface Drain.—The natural ground beyond the downstream toe of the dam shall be dressed off for a width of 30 feet, with an inclination from the dam of 1 in 10 to form a surface drain, which shall be constructed in disconnected sections corresponding with the natural slope of the ground and not exceeding 300 feet in length, and having a longitudinal fall sufficient to carry off the drainage to be dealt with. Each section shall be separated from the one downstream of it by an unexcavated strip not less than 10 feet wide so as to prevent the formation of longitudinal scour

channels The discharge of these sections shall be led by outfall gutters, starting from just upstream of the unexcavated strips, and shall be carried across the downstream drain (clause 16) in watertight channels, and these gutters shall be continued in open excavation to discharge clear of that drain.

16. The Downstream Drain.—Just clear and downstream of the surface drain shall be a downstream drain, which shall have a base width of 5 feet, side-slopes not flatter than 1 in 4, and a continuous longitudinal fall It shall, if possible, be carried 1 foot into sound rock, but, where this does not exist, it shall have a depth of not less than 10 feet Wherever practicable, it shall be constructed in discontinuous sections, discharging by cross drains of similar design into the natural drainage lines, and each of these sections shall be separated from the neighbouring ones by unexcavated strips 10 feet wide

At the base of this trench shall be a drystone slab drain 4 inches wide and 6 inches deep at the head, and gradually increasing in section as it is continued downstream. The trench shall be filled with sound, dry material from the excavations, &c , up to within 3 feet of the ground surface (the larger particles being placed at the bottom), and this material shall be lined at the sides for 1 foot and covered at the top by 2 feet of fine gravel and coarse sand. The whole shall be finished off by a cover of soil carried up to 1 foot above ground level

17. Rear Drains.—At all valley lines crossed by the dam shall be rear drains leading from the puddle trench drain at right angles to the axis of the dam, with as steep longitudinal slopes as practicable, into the natural drainage lines. Their section shall be in proportion to the drainage they will have to discharge. Under the dam, if excavated in soil, they shall be constructed like the downstream drain, but outside the dam, if in rock, they may be left as simple excavations, care being taken to divert all surface flow from them.

The main rear drain shall lead from the deepest part of the foundation where the main stream is crossed by the dam, and shall be constructed with particular care, so that its flow shall

never be obstructed A flushing pool shall be constructed at its head, in order to assist in keeping it clear.

18. General.—The longitudinal and cross-sections of the various drains shall be clearly shown on the drawings, and the drains themselves shall be constructed in strict accordance with them, except when deviations are permitted in writing by the Engineer. The whole of the excavations for the drains shall be inspected by the Engineer, and they shall not be filled in until he has certified in writing that he has passed them.

III. The Superstructure of the Dam

19. The Sections of the Dam.—The sections of the dam shall be clearly shown on the drawings, the work shall be constructed in strict accordance with them, and no deviation from them shall be permitted except with the written permission of the Chief Engineer. These sections shall show the limits of the hearting and the thickness of the casings; the toe and crest walls and the berms, if any; and shall clearly indicate all levels, slopes and dimensions.

20. The Material of the Dam.—The hearting shall consist of a mixture of pure clay and pure grit in the proportion of 1 to 1, but, where these materials are not procurable within half a mile of the site, the best available within that distance shall be used, in such proportions as will result in an equivalent mixture The soils used shall be free from all slushy, salty, sandy, peaty, or powdery material, rubbish and vegetation, and shall be of a tough nature, not liable to become greasy when wet or to crack when dry

The casings shall consist of the same materials as the hearting, but their mixture shall be equivalent to one of 1 part of pure clay to 2 parts of pure grit The whole section of the dam shall be constructed at the same time, and patchings-on shall not be allowed except with the written permission of the Engineer.

21. Spreading and Mixing.—The clay shall first be deposited evenly to the proper thickness on the wetted surface of the last completed layer, and shall be covered evenly with the proper thickness of grit, after all clods and large particles have been broken up. The two shall then be intimately and thoroughly incorporated with each other by hand, by

harrows, or by inverting ploughs, as may be directed, so as to form one homogeneous mixture free from all stratification. If, after the completion of the finished layer, such stratification is detected, the layer shall be excavated, and properly remade After the mixture has been completed, the layer under construction shall be levelled and its surface made uniform by hand or by harrows.

22. Consolidation.—On the completion of the mixture, the layer shall be thoroughly and evenly consolidated by rollers, so that no further compression by them is practicable and so that a loaded cart travelling over the layer shall not make a rut of perceptible depth on it. In places where the roller cannot work, the consolidation shall be effected by uniform ramming, which shall similarly be continued until further compression is not practicable by it The rammers shall work in unison, ramming first on one side of them and then on the other, and shall advance slowly, thus compacting the earthwork and kneading it together. As ramming is inferior to rolling, it shall be limited to as small an area as possible. The layers, when rammed, shall not exceed 3 inches in thickness.

In order to avoid an excessive and sudden amount of settlement, no part of the embankment shall be raised vertically more than 30 feet in one season except with the written permission of the Engineer.

During construction the layers shall be made at least 18 inches wider on each side of the dam than the designed section, and this extra width shall be dressed off after the embankment has been raised at least 5 feet higher than the layer concerned.

23. Watering.—The quantity of water used shall be rigidly limited to the amount just sufficient to unite the old layer with the new one which is to be constructed on it, and the water shall be evenly distributed over the former, just before the construction of the latter is commenced.

Should the new layer during consolidation crack or move in front of the roller, this is an indication that too much water has been used, the new layer shall then be dug up and its material shall be allowed to dry partially, and shall then be levelled and reconsolidated Similarly, should the material be

found on inspection of the weekly trial pits (clause 26) to be too wet, it shall be cut out and remade, unless the Engineer by written order permits it to remain

Should the materials to be brought on to the dam be in too dry a state to be thoroughly consolidated, they shall be watered *in situ*, and shall be utilised when in a proper condition. Soils which are naturally in too damp a condition to be used shall be excavated and allowed to dry *in situ* until they are in a fit state for the work

After the completion of a portion of the embankment and before the final dressing off of the slopes to proper section, in order that the surface may be thoroughly compacted, water shall be poured down the slopes for several days until the earthwork is not able to absorb more moisture

24. Thickness of Layers.—The materials of the layers shall be so deposited in thickness that, when thoroughly consolidated by ordinary rollers, the finished layers shall have a thickness of not more than 5 inches for the top 30 feet in height of the dam, of not more than 4 inches for the next 30 feet, and of not more than 3 inches for the remainder of the base of the dam and for the river crossing

When steam rollers are employed, these thicknesses of the layers may be increased by 50 per cent

25. Slopes of Layers.—The surface of the layers shall be level for a width of about one-sixteenth of the section on each side of the centre line of the dam, on the downstream side, it shall slope up at an inclination not exceeding 1 in 10, and on the upstream side, at an inclination which shall make the upstream edge level with the downstream one

26. Testing the Construction of the Dam.—At the close of each week the work constructed during the week shall be tested by means of small trial pits, 2 feet by 2 feet, which shall be excavated throughout its depth, and any change in construction or alteration of the completed work thus proved to be necessary shall be carried out in accordance with the written instructions of the Engineer

27. Junctions of Earthwork.—Junctions shall be avoided as much as possible, but, where unavoidable, they shall thus be constructed —

(*a*) *Cross-sectional Junctions.*—The loose surface earth of the end slope of the old embankment shall be entirely removed, and that slope shall thereafter be cut into a series of joggles and tongues sloping vertically up it. The excavated surface shall be well wetted and the new earthwork consolidated in intimate union with it.

(*b*) *Longitudinal Junctions.*—All loose surface earth shall be removed, and the solid surface of the old embankment shall be cut into a series of benches of irregular width and depth, which shall be wetted. The new earthwork shall then be constructed in layers sloping steeply on to the old embankment, and shall be consolidated in intimate union with it.

Such junctions shall not be raised more than 20 feet in height in one season, and in the case of (*a*) they shall be broken up into steps not exceeding 10 feet in height and separated by horizontal breaks of not less than 50 feet, over which the subsequent work shall lap by a distance of not less than 50 feet.

(*c*) *Additions to height.*—When an old dam has to be raised, all the loose top surface shall be removed, and one or more key trenches, not less than 4 feet wide at bottom and 3 feet deep, and with slightly sloping sides, shall be excavated parallel to the centre line, and shall be carefully filled with the most retentive material procurable within half a mile of the site, and this shall be thoroughly rammed before the main part of the new earthwork is commenced.

28. Finishing off the Embankment.—The dam shall be constructed to the correct lines, widths and levels, and due allowance for settlement shall be made; its slopes shall be dressed uniformly to 1½ inches extra to the designed width and shall then be rammed to that width. The top shall be properly finished off with a fall of 1 inch to the reservoir, and shall be covered with half an inch of coarse sand, which shall be well rolled in.

During construction arrangements shall be made for the prevention of water lodging on any part of the dam, and of the guttering of the slopes by rainfall; for filling in at once any settlements or cracks that may occur; and, generally, for maintaining the whole structure in thoroughly good order.

The work shall proceed uniformly and regularly in as long continuous lengths as practicable

The downstream slope of the dam shall be turfed, or sown, with fine grass of a binding character, and one which will thrive in this situation. Upstream of the full supply contour of the reservoir and downstream of the downstream drain, the dam shall be enclosed by a good fence or hedge.

All works roads up the dam shall be made with, and not patched on to it, when no longer required they shall be neatly dressed off to its final slope.

29. Excavations for Materials.—Excavation shall not be permitted within a width equal to twice the height of the dam from its downstream toe, nor within one equal to four times the height of the dam from its upstream toe. At the side next the dam the depth of such excavations shall not exceed 5 feet, and shall not be increased therefrom at a greater inclination than 1 in 10. The excavation pits on the downstream side shall not be continuous with each other, so that the formation of scour channels may thus be avoided

The entire stripping off of the water-tight cover of pervious strata shall not be permitted on the reservoir side within a minimum width equal to ten times the height of the dam from its upstream toe.

All excavated pits shall be arranged in neat lines and in blocks to facilitate the record of their measurements.

IV. Pitching.

30. The Extent and Thickness of the Pitching.—The pitching shall extend from and to the levels, and shall be of the thickness shown on the drawings, and everywhere that thickness shall be represented by single through stones of the full depth Whenever practicable, the pitching shall not be constructed until the embankment has had a full year in which to settle

31. The Stone.—The pitching shall consist of sound, hard and durable stone which will not weather and will not deteriorate when exposed to the action of water It shall be roughly hammer-dressed, so as to remove all large projections and so that the stones may meet all round their bases for a

depth at least one-quarter of their height and completely cover and protect the embankment. The stones shall be as regular as possible in horizontal and vertical section, and at least nine-tenths of them shall have a horizontal section at the base of not less than 50 square inches for 12-inch pitching; 65 square inches for 15-inch pitching; 80 square inches for 18-inch pitching; and 100 square inches for 2-feet pitching.

32. Laying.—The stones shall be firmly bedded on a layer of sound, hard muram or quarry spauls, at least 6 inches in thickness, and shall be laid with their broadest ends downwards, so that they may meet all round their bases for a minimum depth equal to one-quarter of their height. They shall be hammer-dressed so as to fit closely to each other at their bases, and shall be malleted securely against each other and on to their seat, so that their bases shall be parallel to the slope of the dam, and so that, when struck by a heavy hammer they shall not be disturbed. The stones shall be laid with the longest dimension of their bases as nearly parallel to the axis of the dam as possible, and shall break joint in every direction, so that long unbroken joints may be avoided. After the laying of the pitching has been inspected and passed, any large spaces between the tops of the stones shall be filled each with a single large chip well driven home, so that the stones may be firmly wedged to each other, and so that the upper surface of the pitching may present a fairly regular appearance free from large interstices. The pitching, thus completed, shall be finally tested and passed.

The face slope of the completed pitching shall be that designed for the dam, and the various thicknesses of the bedding and the stones shall be allowed for accordingly in the earthwork of the embankment.

33. Foundation and Top Courses.—The foundation course shall consist of a line of headers, the depth of which shall not be less than 1 foot in excess of that of the pitching stones abutting on them. They shall be roughly squared and shall be fixed on a bed of spauls in a small trench cut to the extra depth required. Where the ground is soft and liable to erosion by wave action, an apron of large quarry spauls, or medium-

sized rubble, shall be formed over and for at least 5 feet beyond the foundation course so as to protect it.

The top course shall consist of a line of headers, the depth of which shall not be less than 9 inches in excess of that of the pitching at the top, and shall project by that amount beyond the face of the pitching. The headers shall be roughly squared, and shall be fitted together with close joints in one continuous line parallel to the top of the dam.

V. Miscellaneous.

34. The Berm.—The berm shall be constructed in accordance with the drawings at the same time as the main embankment and similarly to it. It shall be founded on a good hard stratum, and its base for at least 2 feet in thickness shall be formed of packed rough rubble and sound stony *débris* from the excavtions: on top of this shall be at least 2 feet of sound clean muram. The rest of the berm shall be made of a mixture of 1 part of pure clay to 2 parts of pure grit, or of the natural soils available in the proportions which will form an equivalent mixture. When there is an excess of stony *débris* from the excavations it may be utilised to form the berm, in which event the larger particles shall be placed nearer the base and the outer slope than the smaller ones. The top of the berm and its outer slope shall, however, be made with a casing at least 2 feet thick, of 1 part of pure clay to 2 parts of pure grit or of an equivalent mixture.

The top of the berm shall have a uniform slope of 1 in 20 extending downwards from the slope of the dam to the outer edge of the berm In it shall be formed two water-tight paved drains, each leading diagonally from 5 feet from where the centre line of the berm meets the slope of the dam to the ends of the outer edge of the berm where it abuts on the natural ground. The bed of each drain shall be at least 1 foot wide and shall rest on retentive clay at least 1 foot thick; it shall be formed of stone slabs breaking joint with each other, and at least 4 inches thick, and extending to at least 3 inches beyond the side walls of the drain which shall consist of a single thickness of the longest stones readily available, and not less than 9 inches high and wide. The interior surface of the drain shall be roughly dressed; the joints of the stones

shall not exceed three-quarters of an inch in thickness and shall be set full in mortar. The outfall of each drain shall be excavated as an open channel, which shall discharge clear of the dam

35. The Drystone Toes.—The drystone toes shall be constructed in accordance with the drawings, and shall be founded on sound rock or hard muram, but where these materials are not near the surface, the ground shall be excavated in sloping benches of at least 5 feet mean depth for the foundation of the toes. The base thus prepared for soft soils shall be covered over with muram at least 2 feet thick, formed in thoroughly consolidated layers not exceeding 4 inches in thickness. Under the upstream toe the muram shall be mixed with an equal volume of clay, but under the downstream toe shall be clean and underlain by a layer of quarry spauls, &c., not less than 18 inches thick, which shall be thoroughly packed and drained When the downstream toe is founded on rock or hard muram, drystone drains each 3 feet by 3 feet in cross section shall be constructed through its base in number sufficient to secure its thorough drainage.

The toes shall be built of good, sound rubble stones of the largest size and as flat-bedded as are readily procurable, and they shall be laid in layers roughly normal to the outer slope of the dam. The stones in each layer shall be firmly bedded in clayey muram, which shall be as stiff as practicable, shall break joint with each other and shall interlock with the stones of the layer below them All projecting corners shall be knocked off by a hammer and the stones made to fit roughly together. The joints between the stones shall be packed with clayey muram well worked into them by short iron bars, and each large interstice shall have a single chip of the largest size practicable driven into it The top of each layer shall be thinly covered with clayey muram just before the stones of the upper layer are built on it, and these shall be laid so as to, interlock and break joint with those of the lower layer Each stone shall be malleted firmly to its bed and the adjacent stones, so that the stone may have a solid bearing on it and them.

The upstream face of the upstream toe shall be formed of

roughly-dressed stones of the largest size readily procurable and not less than 12 inches deep in the work, and these shall be set full in mortar. The downstream face of this toe shall, if thus designed, be formed with a concrete batter wall. The concrete shall be made in accordance with Appendix 18^A, Specification 18. The downsteam face of the downstream toe shall be formed similarly to the upstream face of the upstream toe, but the stones shall be set in clayey muram. As many headers as practicable shall be built in the faces so as to tie them into the hearting.

The core walls, if specified to be built of masonry, shall be constructed of uncoursed rubble in accordance with Appendix 18^A, Specification 21, but if of concrete, in accordance with Specification 18 of that Appendix.

36. The Toe Wall.—The toe wall shall be constructed in accordance with the drawings, and shall be securely founded on concrete (*vide* Appendix 18^A, Specification 18), and the superstructure shall be built of coursed rubble masonry (*vide* Appendix 18^A, Specification 19), or of rough-coursed rubble masonry (*vide* Appendix 18^A, Specification 20) as may be specified The top of the wall shall be formed by a coping of rough-dressed stones projecting 3 inches beyond its downstream face.

At the back of the wall shall be a layer of clean quarry spauls, sound stone *débris*, &c., at least 18 inches thick, and between this and the dam shall be a layer of clean muram at least 12 inches thick. Weep holes shall be left through the wall, so as to discharge the drainage thus collected, and a slab drain shall be built through the wall where it crosses the rear drain of the dam.

37. The Crest Wall.—The crest wall shall be constructed in accordance with the drawings, and shall be securely founded on concrete (*vide* Appendix 18^A, Specification 18), and this shall be protected from wave-wash by an apron of pitching at least 2 feet thick and 5 feet wide The superstructure shall be built of coursed rubble masonry (*vide* Appendix 18^A, Specification 19), or of rough-coursed rubble masonry (*vide* Appendix 18^A, Specification 20) as may be specified. The top of the wall shall be formed by a coping

of rough-dressed stones projecting 3 inches beyond its upstream face. At the back of the wall shall be a layer of clean muram at least 1 foot thick, and narrow weep holes shall be left through the wall so as to discharge the drainage thus collected.

APPENDIX 18A.

SPECIFICATIONS FOR THE WASTE-WEIR AND OUTLET.[1]

(*Vide* Chapter V., paragraph 245, page 347.)

INDEX TO SPECIFICATIONS.

[1] Extracted chiefly from Marryat's "Specifications, Rates, and Notes on Work," with certain alterations and additions.

1. General.—The works are to consist of masonry structures as shown on the drawings, and are to be executed in strict accordance with the plans, estimates, and these specifications, except when written orders by the Engineer permit deviations from them to be made. Only the best materials and the soundest form of construction shall be permitted, and everything shall be executed in a thoroughly workmanlike way to the satisfaction of the Engineer, whose decision as to this shall be final. The rates tendered shall include payment for all labour, materials, construction, and subsidiary work necessary for the execution of the different items specified.

I. Earthwork.

2. Foundation Trenches.

i. *General.*—These shall be taken out to the exact width of the lowest step of the footings the sides shall be left plumb where the nature of the excavation permits this to be done , but they shall be sloped down or shored up carefully where the soil appears treacherous or likely to fall in , and, where necessary, they shall be stepped so that the filling in may rest everywhere on horizontal surfaces.

The foundation course shall be accurately set out on the bed of the trench, which shall first be passed by the Engineer.

ii. *Depth of Trenches*—All foundations exposed to the rush or overfall of water shall, if possible, be taken well into sound rock. If this cannot be met with at a reasonable depth, they shall be taken into sound incompressible soil, extending well below the footings, which will not deteriorate under the action of water. In such cases the foundations shall be specially designed to meet the conditions which exist. Wing walls, and works not liable to be affected by scour, &c., shall have their foundations stepped up as designed ; but, as a rule, the steps shall not exceed 2 feet in height.

iii. *Bottom of Trenches.*—The bottom of the trenches shall be dressed level in cross-section, and, before any concrete or masonry is put in, shall be well watered and thoroughly rammed if of soil. No filling of soil shall be allowed in thus bringing the foundation bed to proper level throughout.

iv. *Refilling Trenches.*—The vacant sides of all foundation trenches shall be refilled to the original surface of the ground

with approved material properly placed. If that is of soil, it shall be formed in regular layers not exceeding 6 inches in thickness, and these shall be well watered and rammed. If that is of boulders, these shall be of the largest size practicable, and shall be fitted together and well packed with large chips so as to be immovable when struck by a heavy hammer.

v. *Spoil.*—No material excavated from foundation trenches, of whatever kind it may be, shall be placed nearer than 4 feet from the outer edges of the excavation.

vi. *Excavation Rate*—If work is done by contract, the Contractor's rates shall include the cost of all shoring, pumping, bailing out or draining; and while the masonry is in progress the excavations shall be kept free of water in such manner as the Engineer may direct.

The rate paid shall include lifting and removing soil to any distance within 50 feet of the centre of the excavation. If the soil be removed to a greater distance, the extra schedule rate for increased lead shall be allowed. The rate shall also include filling in round foundation walls, ramming and securing in the ordinary manner. The measurement of the excavation shall be the exact length and width of the lowest step of the footing, according to the drawings or the instructions of the Engineer, and the depth shall be measured vertically.

vii. *Difficult Foundations.*—Difficult foundations, involving pumping arrangements, coffer-dams, etc., when required, shall form the subject of a special specification to be drawn up with distinct reference to the requirements of each particular case.

3. Foundations in Rock.

i. *Depth of Foundation.*—The foundations shall be carried down to rock well able to bear the weight of the masonry, staunch enough not to leak under the pressure of the water impounded, and of a sound nature which will not deteriorate.

ii. *Foundation Plan.*—A foundation plan shall be carefully prepared and maintained, and this shall show the reduced levels of the whole area of the foundations. For convenience of measurement the quantity of foundation constructional work shall be measured from assumed average level planes,

having as large and regular a superficial area as possible, and these shall be indicated on the foundation plan.

iii. *Blasting and Testing Foundations.*—When the foundations have to be blasted care shall be taken, when nearing the foundation level, that the bed rock is shaken as little as possible, and that all shattered rock is removed with bars or heavy hammers before the filling is commenced. Before construction work is begun the whole area of the foundations shall be tested by striking it with a sledge hammer, and any portions which shake under the blows shall be removed. At intervals of the prepared foundation small trial pits, from 3 to 6 feet deep, shall be blasted to test that the rock is sound underneath it for those depths. If these indicate that it is not sound, the foundation shall be deepened accordingly.

iv. *Preparation of Foundations* —The general surface of the foundations shall be roughened to give a good foothold to the superstructure, and, if necessary, shall have trenches excavated in it parallel to the axis of the work, one of which shall be along its downstream toe, and these shall thereafter be filled with masonry or concrete as may be specified. Immediately before construction work is begun the surface of the foundations shall be cleaned with wire brushes, and shall be thoroughly washed. If the rock is sound, not fissured, and not liable to weather, the surface need only be roughened to give the superstructure a hold. If, however, it is of such a character that it will not stand exposure, the foundation shall be carried well into the rock (from 3 to 6 feet in depth), and its bed at once protected by a covering of concrete or masonry 2 to 3 feet thick carried right over the excavated area. In the case of fissured rock, the fissures, if large, shall be cleared and filled with masonry or concrete, and, if small, shall be grouted with cement mortar until there is no danger of leakage under the base of the work

4. Puddle Filling.—The excavation shall be executed of the width and depth shown on the drawings, or as may be ordered in writing, and shall be filled in general accordance with the specification given in Appendix 18, clauses 10 and 11.

5. Embankment.—Where this forms part of the main

dam it shall be constructed in strict accordance with the specification given in Appendix 18, clauses 13, 14, 20 to 25. Where it forms flank or other embankments subsidiary to the main work it may be constructed of 6 in. layers, well rammed and carefully brought to the designed section. Special care shall be taken to consolidate the bank where it abuts on a masonry work so as to make it in perfect water-tight connection with that. Above high-flood level consolidation may be less thoroughly done.

6. Pitching.—This shall be constructed in general accordance with the specification given in Appendix 18, clauses 30 to 33.

II. Materials for Construction.

7. Portland Cement.

i. *Description*.—The cement shall be obtained from a maker approved by the Engineer. It shall be fresh, but thoroughly air-slaked, and of a grey or greenish-grey colour.

ii. *Tests*.[1]—It shall be tested by such of the following tests as the Engineer may direct:—

(*a*) *Fineness of Grinding*.—The cement shall be prepared only from thoroughly burnt clinker, without any admixtures of under-burnt portions or other substances. Not more than 5 per cent. residue shall remain on a sieve of 5,600 meshes, nor more than 12 per cent. on a sieve of 14,400 meshes to the square inch.

(*b*) *Specific gravity*.—When freshly burnt the cement shall have a specific gravity of not less than 3·14 or more than 3·08 when weathered to 6 degrees.[2]

(*c*) *Chemical Analysis*.—A sample shall not contain more than 1½ per cent. magnesia, 1¾ per cent. sulphuric acid, 1 per cent. carbonic acid, 1 per cent. insoluble residue, nor more than 62 per cent. nor less than 58 per cent. lime.

(*d*) *Tensile Tests*.—Test blocks not less than 1 square inch

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[1] Extracted from *The Builder*, December 28th, 1901. Specification proposed at a meeting of the Architectural Association as a standard specification for important works.

[2] "Weathered to 6 degrees" means that when the cement is mixed for the heating test (clause (*f*) below) it will not show a rise in temperature of more than 6° F.

in cross-section shall be made with 20 per cent. water after the cement has weathered. The mixture shall be placed in moulds without ramming, and kept 1 day in a moist atmosphere of temperature not less than 50° F, and afterwards shall be placed in water of not less than 50° F. Some blocks shall be of neat cement, and some of 1 cement to 3 by weight of standard dry sand; the latter to be mixed with only 10 per cent. of water.

Neat cement blocks to bear per square inch :—

400 lbs. after 7 days, 500 lbs after 14 days; and 600 lbs. after 28 days

Cement and sand blocks to bear per square inch :—

100 lbs. after 7 days; 150 lbs. after 14 days, and 200 lbs. after 28 days

Slabs or cakes shall be made of neat cement with 20 per cent. water, shall be kept in air for 24 hours, and afterwards shall be placed in cold water, which shall be raised to boiling heat, and maintained thereat for 3 hours. After this test the slabs shall not show any sign of warping, checking, or radial cracking. The cake with 20 per cent. water shall harden in not less than 3 hours or more than 7 hours. All cement shall show a uniform growth of strength.

(e) *Adhesive Test.*—A pat of neat cement 3 inches diameter and 1 inch thick shall, after 7 days, adhere firmly to the natural surface of a Welsh slab. The slab shall be soaked in water before the application of the cement, and shall be kept moist during the interval.

(f) *Heating Test.*—A sample of the cement made in a paste with water shall not show a rise of more than 6° F. during one hour after mixing. If it shows a greater rise, the cement is not ready for use or testing.

All cement shall be shot on a perfectly dry floor in a water-tight shed near the site of the works to a depth not greater than 1 foot, shall remain there as long as the Engineer may direct, and shall be turned over from time to time as he may order.[1]

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[1] NOTES.

1. The quantity of water used in mixing cement has a great effect on its tensile strength; consequently the proportions prescribed must be carefully

8. Hydraulic Lime.—Only true hydraulic lime shall be used on works for which the use of cement is not specified. Lime shall be considered hydraulic when it sets under water within 7 days as tested by Vicat's needle.

The lime shall be burnt from such hydraulic limestone as the Engineer may approve. It shall be broken into pieces which can pass through a ring 1½ inch in diameter, and particles with a maximum dimension of less than ½ inch shall be screened out before burning. It shall be carefully freed from earth and impurities and particles containing much sand, and shall be burnt at the site of the works with a sufficiency of good charcoal, coal, or such other fuel as the Engineer may direct.

As soon as the burnt lime has cooled in the kilns it shall be slaked with clean water in such quantities as may at once be required and with the minimum amount of water. It shall thereafter be screened through a sieve having at least 36 meshes [1] to the square inch for ordinary work, or through one having at least 64 meshes to the square inch if required for ashlar or fine work. It shall then be freed from all over-burnt, unburnt and unslaked particles and other impurities,

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adhered to. The method of filling also largely affects the strength; the cement should therefore be pressed, not rammed, into the moulds.

2. The amount of water required depends somewhat on the quality of the cement; hot, quick-setting cements require somewhat more water than slow-setting ones.

3. The fineness of grinding affects both weight and strength. Finely-ground cements are lighter than coarse ground, they are weaker when tested neat, but give better results when mixed with sand

4. Sieves of 5,600 mesh should be made of 40 B.W.G. wire, and sieves of 14,400 mesh of 43 B W G.

5 The section of the test blocks has a great influence on the 7-day tensile test—thus, for a test block 2¼ square inches in cross-section, a test of 306 lbs. per square inch is equal to one of 448 lbs. per square inch for a block of 1 square inch in cross-section.

6. The strength of gauged Portland cement increases with age up to a certain time, good cement never deteriorates The strength of cement decreases rapidly with the proportion of sand mixed with it; its strength appears to increase slightly when the cement is mixed with salt water.

7. Standard sand should be such as will pass through a sieve of 400 meshes to the square inch, and be retained by one of 900 meshes. The old test of a striked bushel is now being gradually abandoned, as the method of filling varies, and the test is thus a misleading one: the specific gravity test has taken its place.

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[1] The finer the lime is ground, the stronger will be the mortar made from it.

and none but the lime thus purified and reduced to powder shall be used.

The lime shall be used as fresh as possible, and, as soon as it is prepared, shall be stacked in heaps under cover to diminish air slaking, and shall be preserved perfectly dry. Any portions that may become wet and partially set or hardened from any cause shall be rejected and removed from the works.

9. Sand.—The sand used in mortar and concrete shall be coarse in texture, clean, sharp, and gritty to the touch, and shall consist of only hard, durable particles. It shall be freed by screening and washing from saltpetre, earth and other impurities, and soft particles which will crush under the mortar mill. Fine drift sand, or such as may be mixed with salts, earth, or organic matter, shall on no account be used.[1]

10. Cement Mortar.[2]—Unless otherwise specified, this shall consist of 3 parts by volume of sand to 1 part by volume of cement These materials shall be carefully measured in boxes of standard size, and shall be intimately mixed together while dry. They shall then be sprinkled with only a sufficiency of water, and shall be thoroughly incorporated together. The mortar shall be used within three hours of its preparation, and any not then used shall be rejected or removed from the works or destroyed.

11. Hydraulic Lime Mortar.—Unless otherwise specified, this shall consist of 1 part by volume of fresh-slaked lime powder to 1½ parts by volume of sand for masonry, and 2 parts by volume of sand for concrete: these materials shall be carefully measured in boxes of standard size. Unless otherwise specified, they shall be then placed dry in an edge mill, which if of stone shall be about 3 feet 6 inches in diameter and about 11 inches in width, and the mill track shall be

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[1] It is of the utmost importance that the sand used for mortar shall be perfectly clean and free from clay or other impurities which might prevent the lime from adhering to it. It is therefore advisable that the sand should be thoroughly washed just before it is required for use.

Very fine sand is objectionable for mortar, as that makes the mortar "short" and diminishes its strength. When, however, very thin joints have to be made, finer sand than what is ordinarily required will have to be used.

[2] Mortar of this description should be used where a quick-setting mortar is required, either for foundations with springs, or where the work will be subjected early to the action of water.

about 13 feet in radius, 12 inches wide at bottom and 12 inches deep.

The freshly-slaked lime shall first be placed in the mill-track and ground by itself for 25 revolutions; then dry sand shall be added and the two ground together for 25 revolutions. Thereafter they shall be wetted with only a sufficiency of water to convert them into stiff paste, and shall be thoroughly ground together for 5 hours or 150 revolutions in the edge mill, water being gradually added as the mortar becomes too stiff

All mortar removed from the mill shall be kept moist and in the shade It shall not be used until 3 hours have elapsed from the completion of the grinding, nor shall it be used after 36 hours have elapsed from that time. None that has become dry or partially set shall be used in the work. Rejected mortar shall be removed from the works or destroyed, unless the Engineer in writing permits it to be re-ground and used for less important work.

From the mortar shall be made briquettes of 2-inch or 3-inch cube, which shall be kept damp in the moulds for 24 hours, and shall then be taken out and covered up with sand kept wet for 24 hours, after which they shall be placed in water. If they maintain their shape and continue to set, the lime will be passed as good hydraulic.

Passed briquettes shall be kept in water for specified periods, and shall then be tested for compressive strength. Briquettes made with 2 of sand to 1 of lime shall bear a pressure of not less than 75 lbs to the square inch at the end of 10 days, of 150 lbs. after 1 month, of 300 lbs. after 3 months, and of 500 lbs after 6 months.

12. Gauged Mortar.—When hydraulic lime mortar will at once be exposed to the action of water which will prevent its setting, it shall be gauged (or mixed) in the proportion of 1 volume of cement to 5 volumes of lime, or as may be specified, the former replacing an equal volume of the latter. The cement shall be added half an hour before the completion of the grinding, and the gauged mortar shall at once be used on the work.[1]

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[1] Mortar should be used as stiff as it can be spread; the joints should always be completely filled by it.

Grout is a very thin liquid mortar, sometimes poured over the courses of

13. Concrete Aggregate.

The metal shall be broken from clean, sound, hard and durable stones, free from earthy matter and all soft surface-scale, and shall pass through a ring 2 inches in diameter.

The gravel shall be of clean, sound, hard, and durable, unbroken stone, free from earthy matter and all soft surface-scale, and shall pass through a ring 2 inches in diameter.

Small aggregate consisting of fine chips, small gravel, pebbles and large sand, all of which are sound, hard, durable and perfectly clean, shall be provided in sufficient quantity to lessen the voids in the large aggregate.

The proportions of the mortar and of the various aggregates shall be determined experimentally,[1] so that all interstices in the latter shall be filled by not more than 50 per cent. of the former.

Stones and boulders shall be embedded only in the concrete of thick walls, and shall be of the largest size which can easily be handled. They shall be of fairly regular shape and of hard and durable material, free from soft surface-scale and earthy matter, shall be clean and well wetted just before they are laid, and shall be embedded solidly in mortar with their broadest ends downwards. They shall project beyond the upper surface of the layer in which they are laid, so as to bond that layer with the one above it. They shall be placed at sufficient intervals apart so as to permit of the concrete being well rammed all round them, and shall first be coated with mortar to make them unite with the concrete. They shall be spaced so as to break joint uniformly in the different layers, and in bulk shall form from $\frac{1}{5}$ to $\frac{1}{3}$ of the whole mass of the concrete.

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masonry or concrete, &c., in order that it may penetrate into empty joints left in consequence of bad workmanship. It may also be necessary to use grout for deep and narrow joints between large stones, or for filling in fissures in rock foundations. It is deficient in strength, (owing to its porous nature when set), and should therefore not be used where it can be avoided.

[1] The volume of these interstices should be ascertained by first thoroughly wetting the aggregate, and then placing it in a water-tight box and noting the quantity of water required to fill the box. It is necessary that the matrix shall be in amount rather more than is sufficient to fill the interstices in the aggregate. Generally, the interstices are equal to from $\frac{1}{3}$ to $\frac{1}{4}$ of the aggregate, if that is all of one size, but may be considerably reduced by using an aggregate composed of stones of varying sizes . in the specification the actual proportions of each size should therefore be clearly stated.

14. Stone.—The stone used shall be heavy, clean, hard, durable, tough, solid, and free from flaws, cracks, soft scale and weathered material, and shall be obtained from quarries approved by the Engineer Each stone shall be laid on its natural quarry bed

15. Water.—The water used shall be clean and free from salt and organic matter All stone and aggregate shall be thoroughly wetted before it is embedded in mortar.[1] The work shall be kept properly wet while it is in progress, and until the mortar is well set. The watering shall at first be done carefully by watering cans fitted with roses, so as not to wash the lime or cement out of the mortar while it is "green". After the mortar has set, it shall be watered more liberally by buckets, etc., and, finally, the upper surface of unfinished courses, when work thereon is not in progress, and the completed work shall be kept flooded with water for as long as possible up to one month after the work was laid. On Sundays and other holidays the work shall be kept watered as above specified, and special labourers shall be employed for this purpose Should the mortar perish through neglect of watering, the work damaged thus shall be pulled down and rebuilt at the Contractor's expense. Should the Contractor fail to water the work to the satisfaction of the Engineer, the latter shall supply the men required to water the work properly and charge the cost thereof to the Contractor.

16. Wooden Shutters.—The planks shall be of the dimensions specified, and shall be cut from timber the source and kind of which shall be approved by the Engineer. That timber shall be of the best quality capable of withstanding wet, well seasoned, felled for not less than two years before use, and free from large or loose knots and from shakes or defects of any kind . heart or sap wood shall be rejected. The edges of the planks shall be planed true and square to make

[1] In using hydraulic limes and cements, it should be remembered that the presence of moisture favours the continuance of the formation of the silicates as commenced in the kiln, and that the setting action of mortars so composed is prematurely stopped if they are allowed to dry too quickly. It is therefore of the utmost importance, especially in hot climates, that the stones embedded in mortar should be thoroughly soaked, so that they cannot absorb moisture from it. Thorough watering further renders the stones clean, and thus permits the mortar to adhere properly to them.

properly water-tight joints, and the planks shall be closely fitted together to form the shutter; when that is passed it shall be tarred two coats or as may be directed.

III. Concrete and Masonry.

17. General.

i. *Uniformity of Construction.*—Where practicable, the whole of the concrete and masonry shall be carried up to one uniform level throughout, but where breaks are unavoidable the junctions shall be made in good long steps so as to prevent the formation of cracks between the old and the new work. All junctions of walls shall be formed at the time the walls are being built, and cross walls shall be carefully bonded into the main walls.

ii. *Battered Walls.*—In all battering, retaining, and breast walls, the beds of the stones and the plane of the courses shall be at right angles to the batter.

iii. *Face Stones and Joints.*—Face stones shall have joints of the specified widths for the specified distance from the face.[1]

The backs of the face stones shall not be in one line, so that the hearting may bond with them as much as possible. Face stones shall not be less than 9 inches deep in the work.

For face joints the mortar shall just fill the spaces between the adjacent stones, and shall not be smeared over these stones, nor shall false joints be made. As soon as the mortar has begun to set, the joint shall be rubbed smooth and hard with a special trowel.[2] Unless distinctly specified, the work shall not be pointed, nor shall the joints be lined with the trowel, nor project beyond the face of the masonry

For wide interior joints mortar shall be economised, and the weight of the work increased, by wedging in vertically the

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[1] Masons sometimes chip off only the edges of the stones to the specified widths of the joint, and do not dress the joint to its full depth . this should be prohibited.

[2] When the joint is thus rubbed, the lime works out to its surface, combines slightly with the iron, and when set forms a very hard skin, having a continuous union with the interior of the joint.

It is desirable to keep the mortar of the top of the course half-an-inch below the top of the stone, and to level it up with mortar when the upper course is being laid. This will keep the lower surface of the joint clean, and will help to key the two courses together.

largest-sized chips which will nearly fill the joint, and these shall be well wetted before they are inserted Not more than one such chip shall be placed in any one joint.

iv. *Backing.*—The backing of thick walls shall be made of fair-sized stones, which shall bond together as much as possible, and especially with the facing. Each stone shall be firmly and thoroughly bedded in the mortar without any vacuities being left

v. *Through Stones and Headers.*—These shall be provided of the specified dimensions, and shall be spaced at the specified intervals. Their ends tailing into the work shall not be greatly less in cross-section than their faces. The stones adjacent to them shall be laid carefully to bond with them in the hearting. The headers in the different courses shall be spaced to break joint well with each other.

vi *Levelling up Courses*—This shall not be effected by means of chips, but where the top of a face stone is irregular, it may be levelled up with fine concrete or coarse mortar just before the laying of the course above it. The tops of the hearting courses shall be made by the tops of the stones forming them, and these shall be fairly level The beds of the stones of the upper course shall fit fairly into the interstices of the lower course.

vii. *Thin Walls.*—In such walls the face stones of the two faces shall bond together as much as possible, and the faces shall be bonded together by through stones. The filling between the faces shall not consist of chips and small stones, but shall be of fair-sized stones, breaking joint with the face stones, or of fine concrete [1] (or coarse mortar) well worked into and consolidated between the facings.

viii. *Dressing.*—All stones shall be dressed off the work Further dressing shall not be allowed after they have been

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[1] This fine concrete hearting is particularly useful when a thin wall, such as of an outlet tower, has to be made water-tight The mortar should consist of $1\frac{1}{2}$ parts of sand to 1 of lime, and can be rendered still more impervious by gauging it with cement. The aggregate should pass through a $\frac{3}{4}$-in ring, and should be of fine chips, large sand, &c. The concrete should be laid in 4-in. layers, lightly rammed and formed quickly one on top of the other to make the course of the required thickness. That course should be kept below the top of the masonry facing, so that there may be a break of joint between the two in order to render the work more staunch

laid in the work ; if it is required, the stones shall be taken off the work, dressed, and then relaid

ix *Setting* —All stones shall be set full in mortar, and shall be laid solidly on their beds and close to each other, and large stones shall for this purpose be well malleted on to their beds and next to each other

x *Pointing*[1]—Pointing shall be avoided as much as possible · it is often an indication of bad or slovenly work. When it is ordered in writing or is specified, it shall, if possible, be done while the mortar in the joints is fresh. The old mortar shall be raked out of the joints at least 1½ inches deep ; the dust shall then be brushed out of the joints and the work well wetted or washed with water The mortar for the pointing shall consist of equal volumes of lime or cement, as may be ordered, and fine sand thoroughly incorporated together. It shall be used quite fresh, and shall be carefully worked in so as to fill the joints completely and no more. After it has begun to set, it shall be rubbed smooth and hard with a special trowel. No false joints shall be made, nor shall the joints be raised beyond the surface of the adjacent stones unless this is specified. The work shall be kept wet for at least three days a ter the pointing is complete and until it is quite set

18. Concrete.

i. *Definition* —The concrete shall consist of an aggregate (Specification 13), joined together by a matrix of mortar made of either cement or hydraulic lime mixed with sand (Specifications 10 or 11).

ii. *Proportions of Materials* —The concrete shall be made in such proportions as shall be specified by the Engineer, due regard being had to the purpose for which the concrete is to be used[2]

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[1] The best form of pointing consists in laying the superior face-mortar at the same time as, and backing it up with, the inferior hearting-mortar, just before the stone is set on the two. The two classes of mortar will thus unite thoroughly together, and will not subsequently separate from each other.

[2] On the large Dhupdál weir (Gokak Canal, Belgaum district, Bombay Presidency) the following were the proportions used, the materials being measured out in open-topped boxes. The weir is about a mile long, and has an average height of about 25 feet and an average mean thickness of about

iii *Mixing*.—The concrete shall be mixed by hand on the work or on a special platform formed with planks, concrete, bricks, or sheet iron, &c., so as to keep the material clean, or, when specified, by a concrete mixer.

When mixed by hand the dry aggregate shall be placed in heaps not exceeding 3 cubic feet in size, and shall be thoroughly mixed dry. It shall then be well watered and to it the mortar shall be added, and the two shall be thoroughly incorporated together by shovels until each particle of aggregate is completely coated with mortar,[1] and the mortar is uniformly distributed throughout the mass. As little water as possible shall be used, and the concrete shall be as stiff

16 feet; it was constructed with masonry facings and concrete hearting, of which latter nearly $1\frac{1}{4}$ million cubic feet were laid.

Materials	Box Measure		Parts by volume
	Internal dimensions Inches	Capacity Cubic feet.	
I. HYDRAULIC MORTAR CONCRETE			
(*a*) Aggregate	24 × 16 × 12	2·66	100
(*b*) Mortar (2 of sand to 1 of lime)	16 × 16 × $8\frac{1}{4}$	1·22	46
II. GAUGED MORTAR CONCRETE			
(*c*) Cement (replacing an equal bulk of lime in the mortar) . .	9 × 8 × 2·8	0·12	$4\frac{1}{2}$
(*a*) and (*b*) Otherwise as in I. . .			
III. PORTLAND CEMENT CONCRETE			
(*a*) Aggregate	24 × 16 × 12	2·66	100 (or 7)
(*b*) Sand . . .	16 × 16 × 12	1·77	66·8 (or 4·66)
(*c*) Cement	12 × $9\frac{1}{2}$ × 6	0·40	15 (or 1)

Another specification for lime concrete is—

Metal or broken stones . .	2 parts	6 parts.
Shingle or large gravel . .	2 parts	
Pebbles or small gravel . .	2 parts	
Mortar (2 of sand to 1 of lime) .	3 parts.	

Another specification for cement concrete is—

Aggregate	5 parts.
Sand	2 parts.
Cement	1 part.

In the Chatham Dockyard Extension Works, the volume of the cement used in thick walls was only one-twelfth that of the walls.

[1] The back strokes of the shovel are particularly useful in forcing the aggregate into the mortar.

as practicable. No more concrete shall be mixed than can at once be laid in place, and when Portland cement is used it shall be mixed only just before the concrete is laid, as the cement begins to set very quickly.

iv. *Laying.*—The concrete shall be carefully deposited on its place, not thrown thereon, in layers not exceeding, when consolidated, 6 inches in depth, and preferably in ones of 4 inches in depth, which shall be constructed one on top of the other to form a single course of as great a thickness as practicable but not exceeding 2 feet. Whenever possible, the top of the concrete course shall be formed half a course below that of the masonry against which it abuts so as to break joint with it in order to secure better bond and greater watertightness. Each layer shall at once be well rammed with heavy iron or wooden rammers for as short a time as possible. The ramming shall not cease until the whole mass is thoroughly consolidated, free from voids throughout and on its surface, but shall on no account be continued after the mortar creams up to the surface or the concrete has begun to set. The concrete shall be a perfectly homogeneous, watertight mass, and shall adhere firmly and solidly to all surfaces against which it abuts, which should for this purpose be cleaned, washed, and plastered with mortar just before the concrete is placed against them. Any portions of the concrete which may become dry or partially set before it is laid or consolidated shall be rejected.

After every course is completed the concrete shall be kept damp and perfectly clean. If more than two days elapse before the next course is laid, the surface of the old one shall then be slightly picked up to secure bond, watered, and grouted All interstices on the surface and sides of the old course shall be grouted with mortar just before the new course is laid on it, and wherever interstices are found unavoidable in the new course, a little additional mortar shall at once be worked in and the whole properly consolidated. Every endeavour shall be made to obtain a perfectly sound, watertight mass. At vertical intervals of from 3 to 5 feet the concrete shall be allowed to set partially, and shall then be flooded with water for at least one day, care being taken that

the flooding is delayed or restricted in amount so as not to wash out the lime [1]

v. *Work included in the Rate.*—Unless otherwise specified, the rates entered on the schedule shall include the cost of mixing, lifting, placing, ramming, and watering, and the provision of wheeling planks, barrows, tools, and all appliances required to complete the concrete in position.

19. Coursed Rubble Masonry.[2]

i. *Height of Courses*—The stones shall be laid in horizontal courses not less than 7 inches in height, and all courses of the same height unless otherwise specified, in which case no course shall be thicker than any course beneath it.

ii. *Dressing.*—The face stones shall be squared on all joints and beds The beds shall be hammer or chisel dressed, true and square, for at least 3 inches back from the face, and the joints for at least $1\frac{1}{2}$ inches. The face of the stones shall be hammer dressed, and "bushing" shall not project more than 1 inch from the face.

iii. *Joints.*—No pinnings shall be allowed on the face. All side joints shall be vertical and beds horizontal, and no joint shall be more than $\frac{3}{8}$ inch in thickness

iv. *Size of Stones*—No face stone shall be less in breadth than its height, nor shall tail into the work to a length less than its height, and at least one-third of the stones shall tail into the work at least twice their height, or in thick walls, three times their height.

v. *Through Stones.*—Through stones shall be inserted from 5 to 6 feet apart in the clear in every course, and shall run right through the wall when it is not more than 2 feet thick. When the wall is thicker, a line of two or more headers or stones shall be laid from face to back and shall overlap each other at least 6 inches.

vi. *Break of Joint.*—Stones shall break joint at least half the height of the course.

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[1] From time to time small trial pits should be excavated in the concrete to test that it has been properly formed and consolidated, and is properly setting.

[2] This class of work is suitable for small weir walls, outlet head walls, lining walls, and for the toe wall and crest wall of the dam embankment.

vii. *Quoins*.—The quoins shall be of the same height as the course in which they occur, and shall be formed of header stones at least $1\frac{1}{2}$ feet long, laid lengthwise alternately along each face. They shall be laid square on their beds, which shall be fairly dressed to a depth of at least 4 inches

viii. *Interior of Wall*.—The interior of the wall shall consist of flat-bedded stones carefully laid on their proper beds and solidly bedded in mortar. Wetted spauls shall be wedged in vertically wherever necessary, and care shall be taken that no dry work or hollow spaces shall be left anywhere in the masonry The face work and backing shall not be levelled up at each course by the use of chips.

ix. *Interior Face*.—The work on the interior face shall be precisely the same as that on the exterior face, unless the former is not exposed to view, in which case the side joints need not be vertical.

20. Rough-coursed Rubble Masonry.[1]

This description of masonry shall be similar to coursed rubble masonry (Specification 19) except that the face stones shall be built in roughly level courses and not in exactly level ones.

21. Uncoursed Rubble Masonry.[2]

i. *Dressing*.—The stones shall be set in the work as received from the quarry, and without further dressing of any sort than that of knocking off weak corners and edges with the mason's hammer.

ii. *Bond and Laying*.—The stones shall be carefully laid so as to break joint as much as possible, and shall be solidly bedded with close joints, none of which shall exceed $\frac{3}{4}$ of an inch in thickness on the face of the wall. Spauls shall be wedged between the backs of the face stones and the hearting stones as may be necessary to avoid thick, vertical

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[1] This class of work is suitable for smaller and cheaper works than those constructed of coursed rubble, and for the upstream casing of the headwall in the centre line of the dam (Specification 26 (v.) below).

[2] This class of work is suitable for low, unimportant walls which have to be constructed cheaply; for the filling of spandrels of arches, and for the hearting of the headwall in the centre line of the dam (Specification 26 (vi.) below).

joints of mortar and to increase the weight of the work. No dry work or hollow spaces shall be allowed anywhere, every stone, whether large or small, shall be set flush in mortar, smaller stones in the filling being carefully selected to fit roughly the interstices between the larger ones

iii. *Hearting Stones*—A fair proportion of the stones used in the hearting shall be of large size Thirty per cent. of them shall exceed three-quarters of a cubic foot in content

iv. *Face Stones.*—The face stones shall be laid, as far as possible, without pinnings in front, and they shall be selected from the mass of quarry stone for greater size, good beds, and uniform colour. They shall be laid so that they shall tail back and bond well into the work, and shall not be of greater height than either their breadth or face, or length of tail in the work Fifty per cent. of these stones shall be of 1 cubic foot in content, and 25 per cent shall be headers tailing into the work at least 15 inches.

v *Through Stones*—One through stone shall be provided for every square yard of facing. It shall be at least half a square foot in area of face, and shall run back into the work at least 2 feet, or be the full depth of the work if that is less than 2 feet. If the wall be over 2 feet thick, a line of headers over-lapping each other 6 inches shall be laid right through the wall.

vi. *Quoins.*—The quoins, unless otherwise specified, shall be of selected stone, neatly dressed with the hammer or chisel to form the required angle, and laid header and stretcher alternately No quoin stone shall be less than 1 cubic foot in content.

22. Ashlar—Rough-tooled.[1]

i. *Dressing.*—The faces exposed to view shall have a fine dressed chisel draft $\frac{3}{4}$ of an inch wide all round the edges, and be rough-tooled between the drafts and on all beds and joints, full, true and out of winding, if the surfaces are plane, or to uniform curves or twists if required by the design. The course

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[1] This class of work is suitable for the sills, lintels, and cut-water caps of large sluices and for the flat pavement of outlet culverts (Chap. IV., para. 202 (*d*), p. 293).

lines shall be truly horizontal and the side joints truly vertical throughout.

ii. *Joints.*—The joints shall be set in ordinary or gauged mortar as may be specified, the beds and joints being in no case more than ¼ inch in thickness, and all visible edges shall be quite free from unsightly chippings. Each stone when laid shall be struck with a maul to bring it to a solid bearing both as to bed and joint.

iii. *Size of Stones.*—The stones shall be laid in regular courses not less than 12 inches in height, and all courses shall be of the same height, unless otherwise specified, in which case no course shall be thicker than any course below it. No stone shall be less in breadth than in height, nor less in length than twice its height.

iv. *Bond.*—The face stones shall be laid header and stretcher alternately, unless otherwise ordered, the headers being arranged to come as nearly as possible in the middle of the stretchers above and below. The stones shall break joint on the face for at least half the height of the course, and the bond shall be carefully maintained throughout the wall.

v. *Through Stones.*—In walls 2½ feet thick and under the headers shall run right through the wall, for thicker walls a line of headers shall be laid from face to back and these stones shall overlap each other at least 6 inches.

vi. *Flat Pavements of Outlet Culverts.*—For these the largest stones easily procurable shall be used and shall be of the depth specified. They shall be set in cement mortar and shall break joint as much as practicable. They shall be laid with their length parallel to the axis of the culvert, with a longitudinal fall, as specified or shown in the drawings, and the side stones shall be well keyed under the arch ring

23. Block-in-Course Facing.[1]

i. *Dressing.*—The face of the stones shall be left rough (but no projection shall exceed 1½ inches) without chisel draft, except at quoins, where a ¾ inch draft shall be given. The interior of the beds and joints shall be rough-tooled without

[1] This class of work is suitable for the facing of important weirs and outlet head walls

projections but the backs of the stones may be left rough, as they come from the quarry.

ii. *Joints.*—The joints and beds of all stones shall be truly vertical and horizontal. The joints shall be rough-tooled true and square for at least 4 inches, and the beds for at least 6 inches from the face, and for these distances the joints shall not exceed ½ inch in thickness. Each stone shall be set full in ordinary mortar and shall be well malleted to bring it to a solid bearing both as to bed and joint.

iii. *Size of Stones.*[1]—The height of the courses shall not be less than 7 inches, and all courses shall be of the same height, unless otherwise specified, in which case no course shall be thicker than any course below it. No stone shall be less in breadth than in height, nor less in length than twice its height, unless otherwise specified.

iv. *Break of Joint.*—Stones shall break joint at least half the height of the course.

v. *Headers* —These shall be spaced 5 feet apart clear, and in each course each shall be spaced one-third of this interval apart from the header below it in the lower course, so that in every fourth course the headers shall be vertically over those of the third course below them. The headers shall run quite through the backing in walls 2½ feet thick and under, and in thicker walls a line of headers shall be laid from back to face, and these stones shall overlap each other at least 6 inches. The backing, if of masonry, shall be carried up simultaneously with the face work.

24. Copings, String Courses and Quoins.

i. *Copings.*—These shall be in as long stones as are easily obtainable, but not less than 18 inches in length, and shall break joint with the stones in the course below.

ii. *String Courses.*—These shall tail into the work to such depth as the Engineer shall direct. The projecting portion only shall be paid for as special work.

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[1] On the Dhupdál weir the height of the courses was uniformly 9 ins , the length of the stone on the face 12 ins., and its depth in the work 16 ins., at 5 feet apart clear were headers 2 ft. 4 ins. long The joints were ½ in. wide for 3 ins. and the beds for 5 ins from the face The minimum break of joint was 5 ins. Concrete was filled in between the masonry facings which were only one stone thick.

iii. *Quoins.*—These shall be laid header and stretcher in alternate courses They shall ordinarily be of the full height of one course, but if so ordered by the Engineer, may be of the height of two courses

iv. *General.*—All these classes of work shall be fine-tooled, rustic-faced, or as may be directed by the Engineer, and shall be dressed exactly to template The stones shall be dressed on all beds, joints and faces full, true and out of winding, if the surfaces are plain, or to uniform curves or twists if required by the design They shall be set in fine mortar, which shall, if directed, be gauged with cement; the beds and joints shall in no case exceed $\frac{1}{4}$ inch in thickness, and all visible edges shall be quite free from unsightly chippings.

Each stone when laid shall be struck with a maul to bring it to a solid bearing both as to bed and joint

All mouldings shall be worked to templates cut out of sheet zinc or tin.

All copings shall, if ordered by the Engineer, be joined together by dowels or cramps, which shall be of the hardest and toughest stone procurable, or of copper, and shall be set in pure cement. Iron cramps shall not be used

25. Block-in-Course Arching.[1]

i. *Arch Stones.*—These shall generally (but see iv., below) be of the entire thickness of the arch, and shall be carefully and accurately wrought to give the proper radiating joints, that is, the arch stones shall be dressed full and true to their proper shapes, with the necessary summering, twist or winding, and shall be carefully set in good fine mortar.[2]

ii. *Dressing Beds*—The intrados, joints, and beds shall be fair-tooled and left full; the last or keying course shall be accurately fitted and driven into its place with heavy wooden beaters.

iii. *Face Stones.*—The face stones shall be tooled, or rock-faced, with or without chamfers or mouldings, as may be specified or shown on the drawings.

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[1] This class of work is suitable for the arches of the weir sluices, and for the ring of the outlet culvert.

[2] See end of footnote No. 2, p. 414.

iv. *Size of Stones.*—The arch stones shall not be less than 7 inches in their least dimension, and shall break joint at least 7 inches In arches up to 2 feet in thickness the stones shall all be of the full thickness of the ring In arches from 2 feet to 3 feet in thickness, the stones shall be laid header and stretcher alternately, all the headers being of the full depth of the ring and not more than two stretchers going to make up the thickness of the ring Exact uniformity will be required in the thickness of each course of arch stones.

v. *Joints*—The bed joint shall be perpendicular to the tangent of the curve of the arch at each joint, the side joint shall be at right angles to the face and bed joints, and the thickness of the joints shall not exceed ⅜ inch.

vi. *Centres*—Arches shall be built on proper centres approved by the Engineer, and no centres shall be eased or struck without his permission. During the progress of the work care shall be taken to distribute the load on the centres in order to obtain a true curve at the completion of the work. The rate for arch work shall include the provision of proper centres and their setting up, easing and removal.

vii. *Removal of Bad Work.*—If any arch settles unduly, or becomes unsightly through carelessness, bad workmanship, or bad material, it shall be removed and rebuilt at the Contractor's expense

viii *Pointing.*—The mortar of the joints on the face or soffit of the arch shall be raked out as soon as the centering is removed and shall be neatly pointed with good mortar or cement (Specification 17 (x)).

ix *Measurement.*—The measurement of the arch work shall be the mean of the lengths of the extrados and intrados, the full breadth of the arch and the full thickness of the stone put into the arch.

26. Headwall in the Centre Line of the Dam.[1]

i. *General Construction.*—The masonry shall consist of heavy, sound, hard and tough stone of a durable nature, thoroughly bedded in mortar, consisting of hydraulic lime or cement and clean sharp sand, and shall be of the following classes :—

[1] This specification is also suitable for large masonry dams.

(a) Downstream casing of block-in-course facing, backed with coarsed rubble masonry ; or it may consist of coursed rubble masonry throughout

(b) Upstream casing of coursed or rough-coursed rubble masonry

(c) Hearting of uncoursed rubble masonry or concrete.

ii. *Face Casings* —The downstream casing shall average 1 foot to 3 feet in thickness, and shall consist of a facing of block-in-course masonry (Specification 23), laid at right angles to the plane of the face, and a backing of coursed rubble masonry (Specification 19), or shall be entirely of coursed rubble masonry as may be specified. The height of the courses shall not be less than 7 inches, and no stone shall be less in length or breadth than 1 foot, or less in breadth than $1\frac{1}{4}$ times its depth

The upstream casing shall be of coursed rubble masonry backed by rough-coursed rubble masonry, or shall be entirely of rough-coursed masonry (Specification 20).

Both the upstream and downstream casings shall be laid carefully to profile and shall be of especially selected stones, carefully fitted together without pinnings on the face.

iii *Joints and Beds in Face Casings.*—The joints of all stones in the facings shall be truly vertical, and if not of rough-coursed masonry, the beds shall be truly normal to the plane of the face, and shall be rough-tooled true and square for at least the same distance in from the face as the thickness of the course in which they occur, but the face and back of the stones may be left rough. All stones shall be bedded and set full in mortar, and each shall be securely driven on to the bed and adjacent stone by a heavy wooden maul.

iv. *Break of Joint.*—The stones in alternate layers shall break joint and bond in every direction, the break of joint on the face being equal at least to half the height of the course.

v. *Coursed or Rough-coursed Rubble Backing.*—The facings shall bond well with the coursed or rough-coursed rubble backing, as the case may be, each stone of which shall be the full depth of the course and shall have parallel beds : the side joints may be of any form. The stones of the backing shall be fitted together as closely as is possible without much

dressing. All stones shall be well wetted just before they are laid in mortar, and shall then be driven with a light mallet down on to and into contact with the adjacent stones. The back of the rubble backing shall be left rough so that the stones may bond into the hearting of the wall.

vi. *Hearting of Uncoursed Rubble Masonry.*—Where the interior of the wall may be subjected to a pressure of more than 60 lbs. to the square inch, the hearting shall be of uncoursed rubble masonry (Specification 21), which shall consist of flat-bedded stones carefully laid on their proper beds and bedded full in mortar. Chips and spauls shall not be put under the stones, but shall be wedged vertically into the side joints. The surface of the rubble shall not be brought to a uniform level, but shall be left rough and uneven, and the stones shall be bedded and driven down on to each other with a light mallet. The hearting shall be laid and bonded closely into the casings as soon as possible, after the latter are built, and shall extend horizontally between the inner edges of the casings.

vii. *Hearting of Concrete.*—Where the interior of the wall will not be subjected to a pressure exceeding 60 lbs. to the square inch, the hearting may be of concrete (Specification 18), and as many large stones (Specification 13) as possible shall be properly embedded in it to increase its weight and to improve its bond. Its bottom layer shall extend horizontally across the whole width of the wall between the casings, *i e.*, the base of the concrete hearting in cross-section shall not be a truncated wedge following the line of the limit of pressure permissible.

viii *Watering.*—All stones used in the work shall be well cleaned and soaked in water before being laid, and all masonry and concrete work shall be kept wet for at least a month or until the next course is laid on it (Specification 15).

IV. Ironwork.

27. Sluice Gates complete.[1]

i. *Extent of Contract.*—The contract includes the provision

[1] Specification for the 7 ft. × 4 ft. 6 ins. sluice gates for the Dhupdāl Storage Reservoir (Plate 11): this is given as a type specification to be modified according to the actual conditions of the work concerned.

of the whole of the finished material, in accordance with the detailed plans and following specifications, for 10 sluice gates with lifting rods and gear complete, and for its conveyance by rail and road to the site of the works and for its erection complete in place.

ii *Sluice Openings*.—The sluice openings are 7 feet by 4 feet 6 inches in the clear and the bottom of the sluice vents is 36 feet below the level of the top of the headwall on which the screw gear bears. The maximum depth of water above the sills of the sluices is 29 feet

iii. *Sluice Gates Frames.*—The frames on which the sluice gates will work are to be of cast-iron as shown on the detailed plan. they are to be 15 feet 2 inches long and generally 5 feet 8 inches wide, and are to be arranged to allow of a vertical travel of the gates of 7 feet 3½ inches from the lowest position. The frame castings are each to be in one piece, and before the gun-metal faces are pinned on are to be planed for their whole length and width over which the gates will bear when closed, or slide when opened If on account of their length the frame castings should not be sufficiently straight when cast to take a firm bearing on the masonry at the back, they are also to be planed at the back to ensure such bearing.

iv. *Sluice Guides.*—The sluice guides are to be of cast iron and are to be 12 inches shorter at the top and 6 inches shorter at the bottom than the sluice gate frames They are to have a planed face for their whole length where they bear upon those frames to which they will be bolted by bolts countersunk at the back as shown on the drawings. They are also to be planed on the face projecting over the gates so as to allow a clearance of only ⅛ inch to the gate

v. *Fixing Sluice Gate Frames*.—Each frame with its guides is to be secured in position on to the masonry by 8 steel wedges (4 on each side of the gate) and by 4 holding-in bolts 1½ inches diameter (2 on each side of the gate) with anchor plates as shown on the plan

vi *Gun-metal Sliding Faces.*—The frames are to be fitted the full length longitudinally at both sides and transversely at the top and bottom of the opening with gun-metal faces 3½

inches wide by $\frac{5}{16}$ inch finished thickness which are to be pinned on to the previously planed cast-iron faces by gun-metal pins $\frac{5}{16}$ inch diameter spaced longitudinally $1\frac{1}{2}$ inches apart zig-zag as shown on the plan. After they have been pinned on, the gun-metal faces are to be finished true by being planed again and then filed and scraped

vii. *Sluice Gates* —The sluice gates are to be of the best cast iron 7 feet 7 inches long by 5 feet 1 inch wide, and are to be of a buckled form stiffened inside by vertical and horizontal ribs The general thickness of the body of the gates is to be 1 inch with bearing faces thickened as shown on the plan. The bearing faces of the gates are first to be planed and then to be fitted with gun-metal faces as described for the frames The gates are to be tried for water-tightness by placing each on its own frame and the faces in sliding contact are to be scraped perfectly true. The back of each gate is to be cast with a cored pillar, as shown on the plan so as to take the lower end of the lifting rod to which the gate is attached. The core is to be slightly oval in section in order to give play in a direction at right angles to the working plane of the gate, so that the water pressure may assist in keeping the gate tight, but there is not to be any appreciable play in a direction parallel to the working plane of the gate—that is to say, the lifting rod will nearly fit the core in that direction.

viii. *Lifting Rods* —The lifting rods will be in two lengths each of which will be of the best mild steel. The lower length is to be 18 feet 3 inches over all by $3\frac{1}{8}$ inches diameter ; it is to have a collar forged solid to bear at the top of the gate, and at the bottom is to be screwed and fitted with a round nut through which a pin is to be passed so as to secure the rod tightly to the gate. The top end will be swelled to $4\frac{1}{2}$ inches diameter and will be turned and planed to the form shown on the plan for the joint with the upper length.

The upper length is to be 21 feet $5\frac{1}{2}$ inches over all, of which the lower 9 feet $11\frac{1}{2}$ inches is to be made 3 inches square and to have its bottom end swelled and machined to form the joint with the lower length. The upper 11 feet 6 inches is to be made $3\frac{7}{8}$ inches outside diameter and for a length of 8 feet

9 inches from the top is to have cut in it a square thread screw of ¾ inch pitch

IX. *Joint of the Lifting Rod.*—The upper and lower lengths of each lifting rod are to be connected by a joint, as detailed on the plan, on which they are shown with their ends swelled to 4½ inches diameter, and planed to allow of them being half-lapped over each other. The ends thus fitted together are to be covered by a cast iron-collar 1 foot 6½ inches long and 8 inches external diameter which is to be bored to fit the turned ends of the rod and to be secured to them by 5 turned bolts of 1⅛ inch diameter driven into holes bored through the collar and ends of the rod at one operation. The joint has been calculated to give to each rod a slightly greater area through any section of it than its ordinary diameter of 3⅛ inches, and the number and size of the bolts have been calculated on the same basis.

X. *Rod Guides.*—Two cast-iron guides, 11 feet 3 inches apart centres, are to be provided for each lifting rod to keep it in a vertical plane. They are each to be bolted by 4 holding-in bolts 2 feet 3 inches long and 1 inch diameter to a block of stone smoothly dressed and accurately set in position. The lower guide is to be bored to 3¼ inches diameter, and its hole will thus be ⅛ inch wider than the rod which passes through it. The upper guide is to have a planed hole 3⅛ inch square to guide the rod and to prevent any torsion from its screw being taken by the rod below this guide.

XI. *Lifting Gear.*—The lifting gear of each gate is to consist of a cast-iron standard bolted to a base plate, both being machined at the joint, with holding-down bolts running right through the masonry to the intrados of the arch below. There are to be four of these bolts each 8 feet 4½ inches long and 1½ inches in diameter, with jibs at the lower ends driven up to two washer plates, each 2 feet 8 inches by 9 inches by ⅝ inch, abutting against the intrados, one on each side of the lifting rod, and the bolts are to have nuts at their upper screwed ends.

The standard is to be turned and bored at the top to receive a gun-metal nut machined all over The nut is to have a single thrust collar 2 inches thick, and is to be secured

to the standard by a cast-iron cap turned and bored to fit and bolted to the standard. One of the bolts is to be specially forged with an eye, through which a chain can be passed and padlocked to prevent the movement when not desired of the four-armed wrought iron spanner, having a radius of 3 feet 9 inches, which is to be supplied to actuate the nut. The gate is to be raised or lowered by manual power applied to this spanner and transmitted through the gun-metal nut to the lifting rod and gate

xii. *Tests*—The following tests are to be made by the Contractor at his own expense:—

(a) Each gate is to be tested by dropping on to it a weight of 1 cwt. three times through a vertical fall of 5 feet on to an area of 16 square inches. The gate is to be supported only on its longitudinal bearing faces when thus tested, and the weight may be dropped on any part of its area clear of the bearing.

(b) Each gate when supported only on its longitudinal bearing faces is to be subjected to a load of 15 tons, which is to be applied on the longitudinal centre line of the back of the gate by means of levers, and will thus in effect be equal to a uniformly distributed load of 30 tons.

(c) Each gate is to move evenly on its own frame and is to be tested before dispatch to see that it does this.

(d) Two sets of rods completely fitted together are to be tested with a compressive stress 50 per cent. in excess of the pressure which could be put on them by eight men turning the four-armed spanner of 3 feet 9 inches radius and nut previously described. If any part of the two sets of rods thus tested proves unsatisfactory, all the remaining rods are to be similarly tested, and any defective lengths there may be shall be rejected and replaced by ones which will stand the test If, however, the first two tests are satisfactory in every way, the remainder of the rods need not be tested.

xiii. *Erection.*—Erection is to include the provision of all tackle of every kind required, and all labour, including the services of a European foreman, and the complete gates are to be handed over in thoroughly satisfactory working order. The masonry work in connection with them will be constructed separately from this contract by the Engineer.

APPENDIX 19.

TABLES OF THE CROSS-SECTIONAL AREAS OF PUDDLE TRENCHES.

Vide Chapter II., paragraph 92, page 129.)

Formula :—$A = (B + S\,D)\,D$,

Where A = the cross-sectional area in square feet ,
B = the bottom-width in feet ,
S = the ratio of the side-slopes to unity ,
D = the depth in feet.

TABLE I.

Bottom-width = 10 feet ; Side-slopes = $\frac{1}{4}$ to 1
$A = (10 + 0{\cdot}25\,D)\,D$ sq. ft

SQUARE FEET

h	Decimals									
t.	0 0	0·1	0 2	0·3	0 4	0 5	0 6	0 7	0·8	0 9
0	0 000	1·002	2 010	3 022	4 040	5 062	6·090	7 122	8 160	9·202
1	10 250	11 302	12·360	13 422	14 490	15 562	16 640	17 722	18 810	19·902
2	21·000	22 103	23 210	24 322	25 440	26 562	27 690	28 822	29 960	31·102
3	32·250	33 402	34 560	35 722	36 890	38 062	39 240	40 422	41 610	42·802
4	44 000	45 202	46 410	47 622	48 840	50 062	51·290	52 522	53 760	55 002
5	56 250	57 502	58·760	60 022	61 290	62 562	63 840	65 122	66 410	67 702
6	69·000	70 302	71 610	72 922	74 240	75 562	76 890	78 222	79·560	80 902
7	82·250	83·602	84 960	86 322	87 690	89 062	90·440	91 822	93 210	94 602
8	96 000	97 402	98 810	100 222	101 640	103 062	104 490	105 922	107 360	108·802
9	110·250	111 702	113 160	114 622	116·090	117 562	119 040	120 522	122 010	123 502
0	125 000	126 502	128 010	129 522	131 040	132 562	134·090	135·622	137 160	138·702
1	140 250	141 802	143 360	144 922	146 490	148 062	149 640	151·222	152 810	154·402
2	156 000	157 602	159 210	160·822	162·440	164·062	165 690	167·322	168·960	170 602
3	172 250	173 902	175 560	177 222	178 890	180 562	182 240	183 922	185 610	187 302
4	189 000	190 702	192 410	194 122	195 840	197 562	199 290	201·022	202·760	204 502
5	206 250	208 002	209 760	211 522	213 290	215 062	216·840	218 622	220 410	222 202
6	224 000	225·802	227·610	229 422	231 240	233 062	234 890	236 722	238·560	240 402
7	242 250	244 102	245·960	247 822	249 690	251 562	253 440	255 322	257·210	259 102
8	261 000	262·902	264 810	266 722	268 640	270 562	272 490	274 422	276 360	278·302
9	280 250	282 202	284 160	286 122	288 090	290 062	292 040	294·022	296·010	298·002
0	300 000	302 002	304·010	306·022	308·040	310 062	312 090	314 122	316 160	318·202
1	320 250	322·302	324·360	326 422	328 490	330·562	332 640	334·722	336 810	338·902
2	341 000	343 102	345 210	347 322	349 440	351 562	353 690	355·822	357·960	360·102
3	362 250	364 402	366 560	368 722	370 890	373 062	375·240	377 422	379 610	381 802
4	384·000	386 202	388·410	390·622	392 840	395 062	397 290	399·522	401·760	404·002
5	406·250	408·502	410 760	413 022	415 290	417 562	419·840	422 122	424·410	426·702
6	429 000	431 302	433 610	435 922	438 240	440 562	442 890	445 222	447·560	449·902
7	452 250	454 602	456 960	459 322	461 690	464 062	466 440	468 822	471·210	473·602
8	476 000	478·402	480 810	483·222	485·640	488 062	490·490	492 922	495 360	497·802
9	500·250	502 702	505 160	507 622	510 090	512 562	515 040	517 522	520·010	522 502
0	525 000	527·502	530·010	532 522	535 040	537 562	540 090	542 622	545·160	547 702
1	550 250	552 802	555 360	557 922	560 490	563 062	565 640	568·222	570·810	573·402
2	576 000	578·602	581 210	583 822	586 440	589·062	591 690	594 322	596 960	599·602
3	602 250	604 902	607 560	610 222	612 890	615·562	618·240	620 922	623·610	626 302
4	629·000	631 702	634 410	637·122	639 840	642·562	645 290	648·022	650·760	653·502
5	656 250	659 002	661·760	664 522	667·290	670 062	672 840	675 622	678 410	681·202
6	684·000	686 802	689 610	692 422	695 240	698 062	700 890	703 722	706·560	709 402
7	712 250	715·102	717 960	720 822	723 690	726 562	729 440	732 322	735 210	738 102
8	741·000	743 902	746 810	749 722	752 640	755·562	758 490	761·422	764·360	767 302
9	770·250	773 202	776 160	779 122	782 090	785 062	788 040	791·022	794·010	797·002
0	800 000	803 002	806 010	809·022	812 040	815·062	818 090	821 122	824·160	827 202

OTE.—For bottom-widths differing from 10 feet, multiply the difference in bottom-width by the depth, and or subtract the product, as the case may be, to or from the tabular quantities.

TABLE II.

Bottom width = 10 feet, Side-slopes = ½ to 1

$$A = (10 + 0{\cdot}5\ D)\ D \text{ sq ft.}$$

SQUARE FEET

Depth in Feet	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	1·005	2·020	3·045	4·080	5·125	6·180	7·245	8·320	9·405
1	10·500	11·605	12·720	13·845	14·980	16·125	17·280	18·445	19·620	20·805
2	22·000	23·205	24·420	25·645	26·880	28·125	29·380	30·645	31·920	33·205
3	34·500	35·805	37·120	38·445	39·780	41·125	42·480	43·845	45·220	46·605
4	48·000	49·405	50·820	52·245	53·680	55·125	56·580	58·045	59·520	61·005
5	62·500	64·005	65·520	67·045	68·580	70·125	71·680	73·245	74·820	76·405
6	78·000	79·605	81·220	82·845	84·480	86·125	87·780	89·445	91·120	92·805
7	94·500	96·205	97·920	99·645	101·380	103·125	104·880	106·645	108·420	110·205
8	112·000	113·805	115·620	117·445	119·280	121·125	122·980	124·845	126·720	128·605
9	130·500	132·405	134·320	136·245	138·180	140·125	142·080	144·045	146·020	148·005
10	150·000	152·005	154·020	156·045	158·080	160·125	162·180	164·245	166·320	168·405
11	170·500	172·605	174·720	176·845	178·980	181·125	183·280	185·445	187·620	189·805
12	192·000	194·205	196·420	198·645	200·880	203·125	205·380	207·645	209·920	212·205
13	214·500	216·805	219·120	221·445	223·780	226·125	228·480	230·845	233·220	235·605
14	238·000	240·405	242·820	245·245	247·680	250·125	252·580	255·045	257·520	260·005
15	262·500	265·005	267·520	270·045	272·580	275·125	277·680	280·245	282·820	285·405
16	288·000	290·605	293·220	295·845	298·480	301·125	303·780	306·445	309·120	311·805
17	314·500	317·205	319·920	322·645	325·380	328·125	330·880	333·645	336·420	339·205
18	342,000	344·805	347·620	350·445	353·280	356·125	358·980	361·845	364·720	367·605
19	370·500	373·405	376·320	379·245	382·180	385·125	388·080	391·045	394·020	397·005
20	400·000	403·005	406·020	409·045	412·080	415·125	418·180	421·245	424·320	427·405
21	430·500	433·605	436·720	439·845	442·980	446·125	449·280	452·445	455·620	458·805
22	462·000	465·205	468·420	471·645	474·880	478·125	481·380	484·645	487·920	491·205
23	494·500	497·805	501·120	504·445	507·780	511·125	514·480	517·845	521·220	524·605
24	528·000	531·405	534·820	538·245	541·680	545·125	548·580	552·045	555·520	559·005
25	562·500	566·005	569·520	573·045	576·580	580·125	583·680	587·245	590·820	594·405
26	598·000	601·605	605·220	608·845	612·480	616·125	619·780	623·445	627·120	630·805
27	634·500	638·205	641·920	645·645	649·380	653·125	656·880	660·645	664·420	668·205
28	672·000	675·805	679·620	683·445	687·280	691·125	694·980	698·845	702·720	706·605
29	710·500	714·405	718·320	722·245	726·180	730·125	734·080	738·045	742·020	746·005
30	750·000	754·005	758·020	762·045	766·080	770·125	774·180	778·245	782·320	786·405
31	790·500	794·605	798·720	802·645	806·980	811·125	815·280	819·445	823·620	827·805
32	832·000	836·205	840·420	844·045	848·880	853·125	857·380	861·645	865·920	870·205
33	874·500	878·805	883·120	887·445	891·780	896·125	900·480	904·845	909·220	913·605
34	918·000	922·405	926·820	931·245	935·680	940·125	944·580	949·045	953·520	958·005
35	962·500	967·005	971·520	976·045	980·580	985·125	989·680	994·245	998·820	1,003·405
36	1,008·000	1,012·605	1,017·220	1,021·845	1,026·480	1,031·125	1,035·780	1,040·445	1,045·120	1,049·805
37	1,054·500	1,059·205	1,063·920	1,068·645	1,073·380	1,078·125	1,082·880	1,087·645	1,092·420	1,097·205
38	1,102·000	1,106·805	1,111·620	1,116·445	1,121·280	1,126·125	1,130·980	1,135·845	1,140·720	1,145·605
39	1,150·500	1,155·405	1,160·320	1,165·245	1,070·180	1,175·125	1,180·090	1,185·045	1,190·020	1,195·005
40	1,200·000	1,205·005	1,210·020	1,215·045	1,220·080	1,225·125	1,230·180	1,235·245	1,240·320	1,245·405

NOTE.—For bottom-widths differing from 10 feet, multiply the difference in bottom-width by the depth, and add or subtract the product, as the case may be, to or from the tabular quantities.

APPENDIX 20.

TABLES OF THE CROSS-SECTIONAL AREAS OF DAM EMBANKMENTS

(*Vide* Chapter II, paragraph 73, page 106)

$$\text{Formula} \text{ —} A = \left\{ T + H\left(\frac{S_1 + S_2}{2}\right) \right\} H$$

Where A = the area of the section in square feet,
T = the top-width of the dam in feet,
S_1, S_2 = the ratios to unity of the side-slopes of the dam;
H = the height of the dam in feet, measured from the cleared foundation to the top of the dam and including the allowance for settlement.

TABLE I.

Top-width, 6 feet, Upstream slope, 2 to 1, Downstream slope, $1\frac{1}{2}$ to 1.

$$A = (6 + 1{\cdot}75\ H)\ H \text{ sq. ft.}$$

SQUARE FEET

eight in 'eet	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·617	1·270	1·957	2·680	3·437	4·230	5·057	5·920	6·817
1	**7·750**	**8·717**	**9·720**	**10·757**	**11·830**	**12·937**	**14·080**	**15·257**	**16·470**	**17·717**
2	19·000	20·317	21·670	23·057	24·480	25·937	27·430	28·957	30·520	32·117
3	**33·750**	**35·417**	**37·120**	**38·857**	**40·630**	**42·437**	**44·280**	**46·157**	**48·070**	**50·017**
4	52·000	54·017	56·070	58·157	60·280	62·437	64·630	66·857	69·120	71·417
5	**73·750**	**76·117**	**78·520**	**80·957**	**83·430**	**85·937**	**88·480**	**91·057**	**93·670**	**96·317**
6	99·000	101·717	104·470	107·257	110·080	112·937	115·830	118·757	121·720	124·717
7	**127·750**	**130·817**	**133·920**	**137·057**	**140·230**	**143·437**	**146·680**	**149·957**	**153·270**	**156·617**
8	160·000	163·417	166·870	170·357	173·880	177·437	181·030	184·657	188·320	192·017
9	**195·750**	**199·517**	**203·320**	**207·157**	**211·030**	**214·937**	**218·880**	**222·857**	**226·870**	**230·917**
10	235·000	239·117	243·270	247·457	251·680	255·937	260·230	204·557	268·920	273·317
11	**277·750**	**282·217**	**286·720**	**291·257**	**295·830**	**300·437**	**305·080**	**309·757**	**314·470**	**319·217**
12	324·000	328·817	333·670	338·557	343·480	348·437	353·430	358·457	363·520	368·617
13	**373·750**	**378·917**	**384·120**	**389·357**	**394·630**	**399·937**	**405·280**	**410·657**	**416·070**	**421·517**
14	427·000	432·517	438·070	443·657	449·280	454·937	460·630	466·357	472·120	477·917
15	**483·750**	**489·617**	**495·520**	**501·457**	**507·430**	**513·437**	**519·480**	**525·557**	**531·670**	**537·817**

NOTE.—For top-widths differing from 6 feet, multiply the difference in top-width by the height, and add or btract the product, as the case may be, to or from the tabular quantities.

TABLE II

Top-width, 6 feet, Upstream slope, $2\frac{1}{2}$ to 1 ; Downstream slope, 2 to 1

$$A = (6 + 2{\cdot}25\ H)\ H \text{ sq ft.}$$

SQUARE FEET.

Height in Feet	Decimals									
	0 0	0 1	0 2	0 3	0 4	0 5	0 6	0 7	0 8	0 9
0	0·000	0·622	1·290	2 002	2·766	3·562	4 410	5 302	6 240	7·222
1	8 250	9 322	10 440	11·602	12 810	14 062	15·360	16 702	18 090	19 522
2	21 000	22·522	24·090	25·702	27·360	29 062	30 810	32·602	34 440	36 322
3	38 250	40 222	42·240	44·302	46 410	48·562	50·760	53 002	55 290	57·622
4	60 000	62·422	64 890	67·402	69 960	72·562	·75 210	77 902	80·640	83·422
5	86 250	89 122	92·040	95 002	98·010	101·062	104·160	107 302	110·490	113·722
6	117 000	120 322	123·690	127·102	130 560	134·062	137 610	141·202	144 840	148·522
7	152·250	156 022	159 840	163 702	167 610	171·562	175·560	179 602	183 690	187 822
8	192 000	196 222	200 490	204 802	209 160	213·562	218·010	222·502	227·040	231 622
9	236 250	240 922	245 640	250 402	255 210	260·062	264 960	269 902	274 890	279 922
10	285 000	290 122	295·290	300 502	305·760	311 062	316 410	321 802	327·240	332·722
11	338 250	343·822	349 440	355·102	360·810	366 562	372 360	378 202	384 080	390 922
12	396 000	402 022	408·090	414 202	420·360	426 562	432 810	439 102	445 440	451·822
13	458·250	464 722	471 240	477 802	484 410	491·062	497 760	504·502	511·290	518·122
14	525 000	531·922	538·890	545 902	552 960	560 062	567 210	574·402	581 640	588 922
15	596 250	603 622	611 040	618 502	626 010	633·562	641·160	648·802	656·490	664 222
16	672·000	679·822	687 690	695 602	703 560	711 562	719·610	727·702	735·840	744·022
17	752 250	760 522	768 840	777 202	785·610	794·062	802 560	811·102	819 690	828·322
18	837 000	845 722	854 490	863·302	872 160	881 062	890·010	899 002	908 040	917·122
19	926·250	935 422	944·640	953 902	963·210	972·562	981 960	991·402	1,000·890	1,010·422
20	1,020·000	1,029 622	1,039 290	1,049·002	1,058·760	1,068 562	1,078 410	1,088·302	1,098 240	1,108·222
21	1,118 250	1,128 322	1,138 440	1,148·602	1,158 810	1,169 062	1,179·360	1,189·702	1,200 090	1,210·522
22	1,221·000	1,231·522	1,242·090	1,252 702	1,263 360	1,274 062	1,284·810	1,295·602	1,306 440	1,317·322
23	1,328 250	1,339·222	1,350 240	1,361·302	1,372·410	1,383 562	1,394 760	1,406·002	1,417 290	1,428·622
24	1,440·000	1,451·422	1,462 890	1,474 402	1,485·960	1,497 562	1,509 210	1,520·902	1,532·640	1,544·422
25	1,556·250	1,568·122	1,580 040	1,592 002	1,604 010	1,616·062	1,628 160	1,640·302	1,652·490	1,664·722

NOTE —For top-widths differing from 6 feet, multiply the difference in top-width by the height, and add or subtract the product, as the case may be, to or from the tabular quantities.

TABLE III.

Top-width, 8 feet; Upstream slope, 3 to 1, Downstream slope, 2 to 1

$$A = (8 + 2{\cdot}5\ H)\ H \text{ sq ft.}$$

Square Feet.

Decimals

0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0·000	0·825	1·700	2·625	3·600	4·625	5·700	6·825	8·000	9·225
10·500	**11·825**	**13·200**	**14·625**	**16·100**	**17·625**	**19·200**	**20·825**	**22·500**	**24·225**
26·000	27·825	29·700	31·625	33·600	35·625	37·700	39·825	42·000	44·225
46·500	**48·825**	**51·200**	**53·625**	**56·100**	**58·625**	**61·200**	**63·825**	**66·500**	**69·225**
72·000	74·825	77·700	80·625	83·600	86·625	89·700	92·825	96·000	99·225
102·500	**105·825**	**109·200**	**112·625**	**116·100**	**119·625**	**123·200**	**126·825**	**130·500**	**134·255**
138·000	141·825	145·700	149·625	153·600	157·625	161·700	165·825	170·000	174·225
178·500	**182·825**	**187·200**	**191·625**	**196·100**	**200·625**	**205·200**	**209·825**	**214·500**	**219·225**
224·000	228·825	233·700	238·625	243·600	248·625	253·700	258·825	264·000	269·225
274·500	**279·825**	**285·200**	**290·625**	**296·100**	**301·625**	**307·200**	**312·825**	**318·500**	**324·225**
330·000	335·825	341·700	347·625	353·600	359·625	365·700	371·825	378·000	384·225
390·500	**396·825**	**403·200**	**409·625**	**416·100**	**422·625**	**429·200**	**435·825**	**442·500**	**449·225**
456·000	462·825	469·700	476·625	483·600	490·625	497·700	504·825	512·000	519·225
526·500	**533·825**	**541·200**	**548·625**	**556·100**	**563·625**	**571·200**	**578·825**	**586·500**	**594·225**
602·000	609·825	617·700	625·625	633·600	641·625	649·700	657·825	666·000	674·225
682·500	**690·825**	**699·200**	**707·625**	**716·100**	**724·625**	**733·200**	**741·825**	**750·500**	**759·225**
768·000	776·825	785·700	794·625	803·600	812·625	821·700	830·825	840·000	849·225
858·500	**867·825**	**877·200**	**886·625**	**896·100**	**905·625**	**915·200**	**924·825**	**934·500**	**944·225**
954·000	963·825	973·700	983·625	993·600	1,003·625	1,013·700	1,023·825	1,034·000	1,044·225
1,054·500	**1,064·825**	**1,075·200**	**1,085·625**	**1,096·100**	**1,106·625**	**1,117·200**	**1,127·825**	**1,138·500**	**1,149·225**
1,160·000	1,170·825	1,181·700	1,192·625	1,203·600	1,214·625	1,225·700	1,236·825	1,248·000	1,259·225
1,270·500	**1,281·825**	**1,293·200**	**1,304·625**	**1,316·100**	**1,327·625**	**1,339·200**	**1,350·825**	**1,362·500**	**1,374·225**
1,386·000	1,397·825	1,409·700	1,421·625	1,433·600	1,445·625	1,457·700	1,469·825	1,482·000	1,494·225
1,506·500	**1,518·825**	**1,531·200**	**1,543·625**	**1,556·100**	**1,568·625**	**1,581·200**	**1,593·825**	**1,606·500**	**1,619·225**
1,632·000	1,644·825	1,657·700	1,670·625	1,683·600	1,696·625	1,709·700	1,722·825	1,736·000	1,749·225
1,762·500	**1,775·825**	**1,789·200**	**1,802·625**	**1,816·100**	**1,829·625**	**1,843·200**	**1,856·825**	**1,870·500**	**1,884·225**
1,898·000	1,911·825	1,925·700	1,939·625	1,953·600	1,967·625	1,981·700	1,995·825	2,010·000	2,024·225
2,038·500	**2,052·825**	**2,067·200**	**2,081·625**	**2,096·100**	**2,110·625**	**2,125·200**	**2,139·825**	**2,154·500**	**2,169·225**
2,184·000	2,198·825	2,213·700	2,228·625	2,243·600	2,258·625	2,273·700	2,288·825	2,304·000	2,319·225
2,334·500	**2,349·825**	**2,365·200**	**2,380·625**	**2,396·100**	**2,411·625**	**2,427·200**	**2,442·825**	**2,458·500**	**2,474·225**
2,490·000	2,505·825	2,521·700	2,537·625	2,553·600	2,569·625	2,585·700	2,601·825	2,618·000	2,634·225
2,650·500	**2,666·825**	**2,683·200**	**2,699·625**	**2,716·100**	**2,732·625**	**2,749·200**	**2,765·825**	**2,782·500**	**2,799·225**
2,816·000	2,832·825	2,849·700	2,866·625	2,883·600	2,900·625	2,917·700	2,934·825	2,952·000	2,969·225
2,986·500	**3,003·825**	**3,021·200**	**3,038·625**	**3,056·100**	**3,073·625**	**3,091·200**	**3,108·825**	**3,126·500**	**3,144·225**
3,162·000	3,179·825	3,197·700	3,215·625	3,233·600	3,251·625	3,269·700	3,287·825	3,306·000	3,324·225
3,342·500	**3,360·825**	**3,379·200**	**3,397·625**	**3,416·100**	**3,434·625**	**3,453·200**	**3,471·825**	**3,490·500**	**3,509·225**
3,528·000	3,546·825	3,565·700	3,584·625	3,603·600	3,622·625	3,641·700	3,660·825	3,680·000	3,699·225
3,718·500	**3,737·825**	**3,757·200**	**3,776·625**	**3,796·100**	**3,815·625**	**3,835·200**	**3,854·825**	**3,874·500**	**3,894·225**
3,914·000	3,933·825	3,953·700	3,973·625	3,993·600	4,013·625	4,033·700	4,053·825	4,074·000	4,094·225
4,114·500	**4,134·825**	**4,155·200**	**4,175·625**	**4,196·100**	**4,216·625**	**4,237·200**	**4,257·825**	**4,278·500**	**4,299·225**
4,320·000	4,340·825	4,361·700	4,382·625	4,403·600	4,424·625	4,445·700	4,466·825	4,488·000	4,509·225
4,530·500	**4,551·825**	**4,573·200**	**4,594·625**	**4,616·100**	**4,637·625**	**4,659·200**	**4,680·825**	**4,702·500**	**4,724·225**
4,746·000	4,767,825	4,789·700	4,811·625	4,833·600	4,855·625	4,877·700	4,899·825	4,922·000	4,944·225
4,966·500	**4,988·825**	**5,011·200**	**5,033·625**	**5,056·100**	**5,078·625**	**5,101·200**	**5,123·825**	**5,146·500**	**5,169·225**
5,192·000	5,214·825	5,237·700	5,260·625	5,283·600	5,306·625	5,329·700	5,352·825	5,370·000	5,399·225
5,422·500	**4,445·825**	**5,469·200**	**5,492·625**	**5,516·100**	**5,539·625**	**5,563·200**	**5,586·825**	**5,610·500**	**5,634·225**
5,658·000	5,681·825	5,705·700	5,729·625	5,753·600	5,777·625	5,801·700	5,825·825	5,850·000	5,874·225
5,898·500	**5,922·825**	**5,947·200**	**5,971·625**	**5,996·100**	**6,020·625**	**6,045·200**	**6,069·825**	**6,094·500**	**6,119·225**
6,144·000	6,168·825	6,193·700	6,218·625	6,243·600	6,268·625	6,293·700	6,318·825	6,344·000	6,369·225
6,394·500	**6,419·825**	**6,445·200**	**6,470·625**	**6,496·100**	**6,521·625**	**6,547·200**	**6,572·825**	**6,598·500**	**6,624·225**
6,650·000	6,675·825	6,701·700	6,727·625	6,753·600	6,779·625	6,805·700	6,831·825	6,858·000	5,884·225

—For top-widths differing from 8 feet, multiply the difference in top-width by the height, and add or subtract the as the case may be, to or from the tabular quantities

TABLE IV.

Top-width, 10 feet, Upstream slope, 3 to 1; Downstream slope, 2 to 1

$$A = (10 + 2{\cdot}5\ H)\ H \text{ sq ft}$$

Square Feet

Height in Feet.	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0 000	1·025	2 100	3 225	4·400	5 625	6 900	8·225	9·600	11·
1	12 500	14·025	15·600	17 225	18·900	20 625	22·400	24 225	26·100	28·
2	30 000	32 025	34 100	36 225	38 400	40 625	42·900	45·225	47 600	50·
3	52 500	55·025	57 600	60 225	62 900	65·625	68·400	71·225	74·100	77·
4	80 000	83 025	86·100	89 225	92 400	95 625	98 900	102·225	105 600	109
5	112 500	116 025	119·600	123·225	126 900	130 625	134·400	138·225	142·100	146·
6	150 000	154 025	158 100	162 225	166·400	170 625	174 900	179 225	183 600	188·
7	192·500	197 025	201·600	206 225	210 900	215 625	220·400	225·225	230·100	235·
8	240·000	245 025	250 100	255 225	260 400	265 625	270 900	276·225	281·600	287·
9	292·500	298·025	303 600	309·225	314·900	320·625	326 400	332·225	338·100	344·
10	350·000	356 025	362 100	368 225	374·400	380 625	386 900	393 225	399·600	406
11	412·500	419 025	425·600	432·225	438 900	445 625	452·400	459 225	466·100	473·
12	480 000	487 025	494·100	501 225	508 400	515·625	522 900	530 225	537·600	545
13	552 500	560 025	567 600	575 225	582·900	590 625	598·400	606·225	614 100	622·
14	630 000	638 025	646·100	654 225	662 400	670 625	678 900	687·225	695·600	704
15	712 500	721·025	729·600	738 225	746·900	755·625	764 400	773·225	782·100	791·
16	800 000	809·025	818 100	827·225	836 400	845 625	854 900	864·225	873·600	883·
17	892·500	902·025	911 600	921 225	930·900	940 625	950·400	960·225	970·100	980·
18	990 000	1,000 025	1,010 100	1,020 225	1,030·400	1,040 625	1,050 900	1,061 225	1,071 600	1,082·
19	1,092 500	1,103·025	1,113·600	1,124 225	1,134 900	1,145 625	1,156·400	1,167·225	1,178·100	1,189·
20	1,200 000	1,211 025	1,222 100	1,233 225	1,244·400	1,255·625	1,266 900	1,278·225	1,289 600	1,301
21	1,312·500	1,324 025	1,335 600	1,347·225	1,358·900	1,370·625	1,382·400	1,394·225	1,406 100	1,418·
22	1,430 000	1,442 025	1,454·100	1,466 225	1,478·400	1,490 625	1,502 900	1,515 225	1,527 600	1,540·
23	1,552·500	1,565 025	1,577 600	1,590 225	1,602·900	1,615 625	1,628·400	1,641·225	1,654·100	1,667·
24	1,680 000	1,693·025	1,706·100	1,719 225	1,732 400	1,745·625	1,758·900	1,772·225	1,785 600	1,799·
25	1,812·500	1,826·025	1,839 600	1,853·225	1,866·900	1,880·625	1,894 400	1,908·225	1,922 100	1,936·
26	1,950·000	1,964·025	1,978·100	1,992 225	2,006 400	2,020 625	2,034 900	2,049·225	2,063 600	2,078
27	2,092·500	2,107·025	2,121 600	2,136 225	2,150·900	2,165 625	2,180·400	2,195·225	2,210·100	2,225·
28	2,240·000	2,255 025	2,270 100	2,285 225	2,300 400	2,315 625	2,330 900	2,346 225	2,361·600	2,377·
29	2,392·500	2,408·025	2,423·600	2,439 225	2,454·900	2,470 625	2,486·400	2,502 225	2,518·100	2,534·
30	2,550 000	2,566·025	2,582 100	2,598·225	2,614 400	2,630 625	2,646 900	2,663,225	2,679 600	2,696·
31	2,712·500	2,729 025	2,745·600	2,762 225	2,778·900	2,795·625	2,812·400	2,829·225	2,846·100	2,863·
32	2,880 000	2,897 025	2,914 100	2,931 225	2,948 400	2,965 625	2,982·900	3,000 225	3,017·600	3,035·
33	3,052 500	3,070·025	3,087·600	3,105·225	3,122·900	3,140 625	3,158·400	3,176·225	3,194·100	3,212·
34	3,230·000	3,248·025	3,266·100	3,284 225	3,302·400	3,320 625	3,338·900	3,357 225	3,375 600	3,394 0
35	3,412·500	3,431·025	3,449·600	3,468·225	3,486·900	3,505·625	3·524·400	3,543·225	3,562·100	3,581·0
36	3,600·000	3,619 025	3,638·100	3,657 225	3,676 400	3,695 625	3,714·900	3,734 225	3,753 600	3,773 0
37	3,792·500	3,812·025	3,831 600	3,851 225	3,870 900	3,890·625	3,910·400	3,930 225	3,950·100	3,970 0
38	3,990 000	4,010 025	4,030·100	4,050 225	4,070 400	4,090 625	4,110·900	4,131 225	4,151·600	4,172·0
39	4,192·500	4,213 025	4,233·600	4,254·225	4,274 900	4,295 625	4,316·400	4,337·225	4,358 100	4,379 0
40	4,400 000	4,421 025	4,442 100	4,463 225	4,484 400	4,505 625	4,526·900	4,548 225	4,569·600	4,591 0
41	4,612·500	4,634·025	4,655·600	4,677·225	4,698·900	4,720·625	4,742·400	4,764·225	4,786·100	4,808·0
42	6,830·000	4,852·025	4,874·100	4,896·225	4,918·400	4,940 625	4,962 900	4,985·225	5,007 600	5,030 0
43	5,052 500	5,075 025	5,097·600	5,120·225	5,142·900	5,165 625	5,188·400	5,211·225	5,234 100	5,257,0
44	5,280 000	5,303 025	5,326·100	5,349 225	5,372·400	5,395 625	5,418 900	5,442 225	5,465·600	5,489 0
45	5,512·500	5,536·025	5,559·600	5,583·225	5,606·900	5,630 625	5,654·400	5,678·225	5,702·100	5,726·0
46	5,750 000	5,774 025	5,798·100	5,822 225	5,846 400	5,870·625	5,894 900	5,919 225	5,943·600	5,968 0
47	5,992·500	6,017 025	6,041 600	6,066·225	6,090 900	6,115·625	6,140·400	6,165·225	6,190 100	6,215 0
48	6,240·000	6,265·025	6,290 100	6,315 225	6,340·400	6,365·625	6,390 900	6,416·225	6,441·600	6,467 0
49	6,292 500	6,518 025	6,543·600	6,569·225	6,594·900	6,620·625	6,646 400	6,672·225	6,698·100	6,724·0
50	6,750·000	6,776·025	6,802 100	6,828·225	6,854·400	6,880 625	6,906·900	6,933 225	6,959 600	6,986 0
51	7,012 500	7,039·025	7,065·600	7,092·225	7,118 900	7,145 625	7,172 400	7,199 225	7,226·100	7,253 0
52	7,280·000	7,307·025	7,334 100	7,361 225	7,388·400	7,415·625	7,442 900	7,470 225	7,497 600	7,525 0
53	7,552·500	7,580·025	7,607 600	7,635·225	7,662·900	7,690·625	7,718·400	7,746 225	7,774·100	7,802·0
54	7,830 000	7,858·025	7,886,100	7,914 225	7,942·400	7,970·625	7,998·900	8,027·225	8,055 600	8,084·0

Table IV.—*continued.*

Square Feet

Decimals

0 0	0 1	0 2	0 3	0 4	0 5	0 6	0 7	0 8	0 9
8,112 500	**8,141 025**	**8,169·600**	**8,198·225**	**8,226 900**	**8,255 625**	**8,284 400**	**8,313·225**	**8,342·100**	**8,371·025**
8,400 000	8,429 025	8,458 100	8,487·225	8,516 400	8,545 625	8,574 900	8 604 225	8,633·600	8,663 025
8,692·500	**8,722·025**	**8,751·600**	**8,781·225**	**8,810 900**	**8,840 625**	**8,870 400**	**8,900 225**	**8,930·100**	**8,960·025**
8,990 000	9,020 025	9,050 100	9,080·225	9,110 400	9,140·625	9,170 900	9,201·225	9,231·600	9,262 025
9,292 500	**9,323·025**	**9,353 600**	**9,384·225**	**9,414·900**	**9,445·625**	**9,476·400**	**9,507·225**	**9,538·100**	**9,569·025**
9,600 000	9,631·025	9,662 100	9,693·225	9,724 400	9,755 625	9,786 900	9,818 225	9,849 600	9,881·025
9,912 500	**9,944·025**	**9,975·600**	**10,007·225**	**10,038 900**	**10,070 625**	**10,102·400**	**10,134 225**	**10,166·100**	**10 198·025**
10,230·000	10,262·025	10,294 100	10,326·225	10,358·400	10 390,625	10,422 900	10,455·225	10,487·600	10,520·025
10,552 500	**10,585·025**	**10,617·600**	**10,650·225**	**10,682·900**	**10,715 625**	**10,748·400**	**10,781,225**	**10,814·100**	**10,847·025**
10,880 000	10,913 025	10,946·100	10,979 225	11,012 400	11,045 625	11,078 900	11,112 225	11,145·600	11,179 025
11,212 500	**11,246·025**	**11,279·600**	**11,313 225**	**11,346 900**	**11,380 625**	**11,414 400**	**11,448 225**	**11,482·100**	**11,516 025**
11,550 000	11,584 025	11,618 100	11,652 225	11,686·400	11,720·625	11,754 900	11,789 225	11,823 600	11,858 025
11,892·500	**11,927·025**	**11,961·600**	**11,996 225**	**12,030 900**	**12,065·625**	**12,100·400**	**12,135·225**	**12,170·100**	**12,205·025**
12,240 000	12,275 025	12,310·100	12,345 225	12,380 400	12,415 625	12,450 900	12,486 225	12,521·600	12,557,025
12,592·500	**12,628·025**	**12,663·600**	**12,699 225**	**12,734·900**	**12,770 625**	**12,806 400**	**12,842 225**	**12,878·100**	**12,914·025**
12,950 000	12,986 025	13,022·100	13,058 225	13,094·400	13,130 625	13,166 900	13,203 225	13,239 600	13,276·025
13,312 500	**13,349·025**	**13,385 600**	**13,422·225**	**13,458 900**	**13,495 625**	**13,532 400**	**13,569·225**	**13,606·100**	**13,643·025**
13,680·000	13,717 025	13,754·100	13,791 225	13,828 400	13,865·625	13,902 900	13,940 225	13,977 600	14,015 025
14,052 500	**14,090 025**	**14,127·600**	**14,165 225**	**14,202·900**	**14,240 625**	**14,278 400**	**14,316·225**	**14,354,100**	**14,392·025**
14,430 000	14,468·025	14,506·100	14,544 225	14,382·400	14,620 625	14,658 900	14,697 225	14,735 600	14,774 025
14,812·500	**14,851 025**	**14,889·600**	**14,928 225**	**14,966·900**	**15,005 625**	**15,044 400**	**15,083·225**	**15,122·100**	**15,161·025**
15,200·000	15,239 025	15,278 100	15,317 225	15,356·400	15,395 625	15,434 900	15,474 225	15,513 600	15,553·025
15,592 500	**15,632 025**	**15,671·600**	**15,711·225**	**15,750·900**	**15,790·625**	**15,830·400**	**15,870 225**	**15,910 100**	**15,950 025**
15·990 000	16,030 025	16,070 100	16,110 225	16,150 400	16,190 625	16,230·900	16,271·225	16,311·600	16,352 025
16,392 500	**16,433 025**	**16,473·600**	**16,514 225**	**16,554·900**	**16,595 625**	**16,636 400**	**16,677 225**	**16,718 100**	**16,759 025**
16,800 000	18,841 025	16,882 100	16,923·225	16,964 400	17,005 625	17,046 900	17,088·225	17,129 600	17,171 025

—For top-widths differing from 10 feet, multiply the difference in top width by the height, and add or subtract the as the case may be, to or from the tabular quantities

Form of Measurement Sheet for the Estimate of the Embankment of a Dam

1	2	3	4	5	6	7	8	9	10
Point on Section	Height of Dam as per Section	Depth allowed for Clearance	Total (2) × (3)	Allowance for Settlement, $\frac{1}{30}$ × Col 4	Grand total height (4) + (5)	Cross-sectional Area corresponding to (6)	Mean Cross sectional Area	Length between Points on Section	Quantity between Points on Section, (8) × (9)
ft	ft	ft	ft	ft	ft.	sq. ft	sq ft	ft	cub ft.

APPENDIX 21.

TABLES OF THE CROSS-SECTIONAL LENGTHS OF PITCHING.

(*Vide* Chapter II., paragraph 152, page 203.)

Formula :—L = S H.

Where L is the cross-sectional length in feet, and
S, the ratio of the length of the upstream slope to unity,
H, the vertical height.

Table I.

Upstream slope = $1\frac{1}{2}$ to 1, S = 1·803, L = 1·803 H ft.

Feet.

Height in Feet	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·180	0·361	0·541	0·721	0·901	1·082	1·262	1·442	1·623
1	**1·803**	**1·983**	**2·164**	**2·344**	**2·524**	**2·704**	**2·885**	**3·065**	**3·245**	**3·426**
2	3·606	3·786	3·967	4·147	4·327	4·507	4·688	4·868	5·048	5·229
3	**5·409**	**5·589**	**5·770**	**5·950**	**6·130**	**6·310**	**6·491**	**6·671**	**6·851**	**7·032**
4	7·212	7·392	7·573	7·753	7·933	8·113	8·294	8·474	8·654	8·835
5	**9·015**	**9·195**	**9·376**	**9·556**	**9·736**	**9·916**	**10·097**	**10·277**	**10·457**	**10·638**
6	10·818	10·998	11·179	11·359	11·539	11·719	11·900	12·080	12·260	12·441
7	**12·621**	**12·801**	**12·982**	**13·162**	**13·342**	**13·522**	**13·703**	**13·883**	**14·063**	**14·244**
8	14·424	14·604	14·785	14·965	15·145	15·325	15·506	15·686	15·866	16·047
9	**16·227**	**16·407**	**16·588**	**16·768**	**16·948**	**17·128**	**17·309**	**17·489**	**17·669**	**17·850**
10	18·030	18·210	18·391	18·571	18·751	18·931	19·112	19·292	19·472	19·653

Table II.

Upstream slope = 2 to 1; S = 2·236; L = 2·236 H ft.

Feet.

Height in Feet	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·224	0·447	0·671	0·894	1·118	1·342	1·565	1·789	2·012
1	**2·236**	**2·460**	**2·683**	**2·907**	**3·130**	**3·354**	**3·578**	**3·801**	**4·025**	**4·248**
2	4·472	4·696	4·919	5·143	5·366	5·590	5·814	6·037	6·261	6·484
3	**6·708**	**6·932**	**7·155**	**7·379**	**7·602**	**7·826**	**8·050**	**8·273**	**8·497**	**8·720**
4	8·944	9·168	9·391	9·615	9·838	10·062	10·286	10·509	10·733	10·956
5	**11·180**	**11·404**	**11·627**	**11·851**	**12·074**	**12·298**	**12·522**	**12·745**	**12·969**	**13·192**
6	13·416	13·640	13·863	14·087	14·310	14·534	14·758	14·981	15·205	15·428
7	**15·652**	**15·876**	**16·099**	**16·323**	**16·546**	**16·770**	**16·994**	**17·217**	**17·441**	**17·664**
8	17·888	18·112	18·335	18·559	18·782	19·006	19·230	19·453	19·677	19·900
9	**20·124**	**20·348**	**20·571**	**20·795**	**21·018**	**21·242**	**21·466**	**21·689**	**21·913**	**22·136**
10	22·360	22·584	22·807	23·031	23·254	23·478	23·702	23·925	24·149	24·372
11	**24·596**	**24·820**	**25·043**	**25·267**	**25·490**	**25·714**	**25·938**	**26·161**	**26·385**	**26·608**
12	26·832	27·056	27·279	27·503	27·726	27·950	28·174	28·397	28·621	28·844
13	**29·068**	**29·292**	**29·515**	**29·739**	**29·962**	**30·186**	**30·410**	**30·633**	**30·857**	**31·080**
14	31·304	31·528	31·751	31·975	32·198	32·422	32·646	32·869	33·093	33·316
15	**33·540**	**33·764**	**33·987**	**34·211**	**34·434**	**34·658**	**34·882**	**35·105**	**35·329**	**35·552**

Table III.

Upstream slope = $2\frac{1}{2}$ to 1 ; S = 2·693 ; L = 2·693 H ft.

Feet.

Height in feet	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·269	0·539	0·808	1·077	1·346	1·616	1·885	2·154	2·424
1	2·693	2·962	3·232	3·501	3·770	4·039	4·309	4·578	4·847	5·117
2	5·386	5·655	5·925	6·194	6·463	6·732	7·002	7·271	7·540	7·810
3	8·079	8·348	8·618	8·887	9·156	9·425	9·695	9·964	10·233	10·503
4	10·772	11·041	11·311	11·580	11·849	12·118	12·388	12·657	12·926	13·196
5	13·465	13·734	14·004	14·273	14·542	14·811	15·081	15·350	15·619	15·889
6	16·158	16·427	16·697	16·966	17·235	17·504	17·774	18·043	18·312	18·582
7	18·851	19·120	19·390	19·659	19·928	20·197	20·467	20·736	21·005	21·275
8	21·544	21·813	22·083	22·352	22·621	22·890	23·160	23·429	23·698	23·968
9	24·237	24·506	24·776	25·045	25·314	25·583	25·853	26·122	26·391	26·661
10	26·930	27·199	27·469	27·738	28·007	28·276	28·546	28·815	29·084	29·354
11	29·623	29·892	30·162	30·431	30·700	30·969	31·239	31·508	31·777	32·047
12	32·316	32·585	32·855	33·124	33·393	33·662	33·932	34·201	34·470	34·740
13	35·009	35·278	35·548	35·817	36·086	36·355	36·625	36·894	37·163	37·433
14	37·702	37·971	38·241	38·510	38·779	39·048	39·318	39·587	39·856	40·126
15	40·395	40·664	40·934	41·203	41·472	41·741	42·011	42·280	42·549	42·819
16	43·088	43·357	43·627	43·896	44·165	44·434	44·704	44·973	45·242	45·512
17	45·781	46·050	46·320	46·589	46·858	47·127	47·397	47·666	47·935	48·205
18	48·474	48·743	49·013	49·282	49·551	49·820	50·090	50·359	50·628	50·898
19	51·167	51·436	51·706	51·975	52·244	52·513	52·783	53·052	53·321	53·591
20	53·860	54·129	54·399	54·668	54·937	55·206	55·476	55·745	56·014	56·284
21	56·553	56·822	57·092	57·361	57·630	57·899	58·169	58·438	58·707	58·977
22	59·246	59·515	59·785	60·054	60·323	60·592	60·862	61·131	61·400	61·670
23	61·939	62·208	62·478	62·747	63·016	63·285	63·555	63·824	64·093	64·363
24	64·632	64·901	65·171	65·440	65·709	65·978	66·248	66·517	66·786	67·056
25	67·325	67·594	67·864	68·133	68·402	68·671	68·941	69·210	69·479	69·749

Table IV.

Upstream slope = 3 to 1 , S = 3·162 ; L = 3·162 H ft.

Feet.

Height in feet	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·316	0·632	0·949	1·265	1·581	1·897	2·213	2·530	2·846
1	3·162	3·478	3·794	4·111	4·427	4·743	5·059	5·375	5·692	6·008
2	6·324	6·640	6·956	7·273	7·589	7·905	8·221	8·537	8·854	9·170
3	9·486	9·802	10·118	10·435	10·751	11·067	11·383	11·699	12·016	12·332
4	12·648	12·964	13·280	13·597	13·913	14·229	14·545	14·861	15·178	15·494
5	15·810	16·126	16·442	16·759	17·075	17·391	17·707	18·023	18·340	18·656
6	18·972	19·288	19·604	19·921	20·237	20·553	20·869	21·185	21·502	21·818
7	22·134	22·450	22·766	23·083	23·399	23·715	24·031	24·347	24·664	24·980
8	25·296	25·612	25·928	26·245	26·561	26·877	27·193	27·509	27·826	28·142
9	28·458	28·774	29·090	29·407	29·723	30·039	30·355	30·671	30·988	31·304
10	31·620	31·936	32·252	32·569	32·885	33·201	33·517	33·833	34·150	34·466
11	34·782	35·098	35·414	35·731	36·047	36·363	36·679	36·995	37·312	37·628
12	37·944	38·260	38·576	38·893	39·209	39·525	39·841	40·157	40·474	40·790
13	41·106	41·422	41·738	42·055	42·371	42·687	43·003	43·319	43·636	43·952
14	44·268	44·584	44·900	45·217	45·533	45·849	46·165	46·481	46·798	47·114
15	47·430	47·746	48·062	48·379	48·695	49·011	49·327	49·643	49·960	50·276
16	50·592	50·908	51·224	51·541	51·857	52·173	52·489	52·805	53·122	53·438
17	53·754	54·070	54·386	54·703	55·019	55·335	55·651	55·967	56·284	56·600
18	56·916	57·232	57·548	57·865	58·181	58·497	58·813	59·129	59·446	59·762

TABLE IV.—*continued.*

FEET.

Height in Feet	Decimals									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
19	60·078	60·394	60·710	61·027	61·343	61·659	61·975	62·291	62·608	62·924
20	63·240	63·556	63·872	64·189	64·505	64·821	65·137	65·453	65·770	66·086
21	66·402	66·718	67·034	67·351	67·667	67·983	68·299	68·615	68·932	69·248
22	69·564	69·880	70·196	70·513	70·829	71·145	71·461	71·777	72·094	72·410
23	72·726	73·042	73·358	73·675	73·991	74·307	74·623	74·939	75·256	75·572
24	75·888	76·204	76·520	76·837	77·153	77·469	77·785	78·101	78·418	78·734
25	79·050	79·366	79·682	79·999	80·315	80·631	80·947	81·263	81·580	81·896
26	82·212	82·528	82·844	83·161	83·477	83·793	84·109	84·425	84·742	85·058
27	85·374	85·690	86·006	86·323	86·639	86·955	87·271	87·587	87·904	88·220
28	88·536	88·852	89·168	89·485	89·801	90·117	90·433	90·749	91·066	91·382
29	91·698	92·014	92·330	92·647	92·963	93·279	93·595	93·911	94·228	94·544
30	94·860	95·176	95·492	95·809	96·125	96·441	96·757	97·073	97·390	97·706
31	98·022	98·338	98·654	98·971	99·287	99·603	99·919	100·235	100·552	100·868
32	101·184	101·500	101·816	102·133	102·449	102·765	103·081	103·397	103·714	104·030
33	104·346	104·662	104·978	105·295	105·611	105·927	106·243	106·559	106·876	107·192
34	107·508	107·824	108·140	108·457	108·773	109·089	109·405	109·721	110·038	110·354
35	110·670	110·986	111·302	111·619	111·935	112·251	112·567	112·883	113·200	113·516
36	113·832	114·148	114·464	114·781	115·097	115·413	115·729	116·045	116·362	116·678
37	116·994	117·310	117·626	117·943	118·259	118·575	118·891	119·207	119·524	119·840
38	120·156	120·472	120·788	121·105	121·421	121·737	122·053	122·369	122·686	123·002
39	123·318	123·634	123·950	124·267	124·583	124·899	125·215	125·531	125·848	126·164
40	126·480	126·796	127·112	127·429	127·745	128·061	128·377	128·693	129·010	129·326
41	129·642	129·958	130·274	130·591	130·907	131·223	131·539	131·855	132·172	132·488
42	132·804	133·120	133·436	133·753	134·069	134·385	134·701	135·017	135·334	135·650
43	135·966	136·282	136·598	136·915	137·231	137·547	137·863	138·179	138·496	138·812
44	139·128	139·444	139·760	140·077	140·393	140·709	141·025	141·341	141·658	141·974
45	142·290	142·606	142·922	143·239	143·555	143·871	144·187	144·503	144·820	145·136
46	145·452	145·768	146·084	146·401	146·717	147·033	147·349	147·665	147·982	148·298
47	148·614	148·930	149·246	149·563	149·879	150·195	150·511	150·827	151·144	151·460
48	151·776	152·092	152·408	152·725	153·041	153·357	153·673	153·989	154·306	154·622
49	154·938	155·254	155·570	155·887	156·203	156·519	156·835	157·151	157·468	157·784
50	158·100	158·416	158·732	159·049	159·365	159·681	159·997	160·313	160·630	160·946
51	161·262	161·578	161·894	162·211	162·527	162·843	163·159	163·475	163·792	164·108
52	164·424	164·740	165·056	165·373	165·689	166·005	166·321	166·637	166·954	167·270
53	167·586	167·902	168·218	168·535	168·851	169·167	169·483	169·799	170·116	170·432
54	170·748	171·064	171·380	171·697	172·013	172·329	172·645	172·961	173·278	173·594
55	173·910	174·226	174·542	174·859	175·175	175·491	175·807	176·123	176·440	176·756
56	177·072	177·388	177·704	178·021	178·337	178·653	178·969	179·285	179·602	179·918
57	180·234	180·550	180·866	181·183	181·499	181·815	182·131	182·447	182·764	183·080
58	183·396	183·712	184·028	184·345	184·661	184·977	185·293	185·609	185·926	186·242
59	186·558	186·874	187·190	187·507	187·823	188·139	188·455	188·771	189·088	189·404
60	189·720	190·036	190·352	190·669	190·985	191·301	191·617	191·933	192·250	192·566
61	192·882	193·198	193·514	193·831	194·147	194·463	194·779	195·095	195·412	195·728
62	196·044	196·360	196·676	196·993	197·309	197·625	197·941	198·257	198·574	198·890
63	199·206	199·522	199·838	200·155	200·471	200·787	201·103	201·419	201·736	202·052
64	202·368	202·684	203·000	203·317	203·633	203·949	204·265	204·581	204·898	205·214
65	205·530	205·846	206·162	206·479	206·795	207·111	207·427	207·743	208·060	208·376
66	208·692	209·008	209·324	209·641	209·957	210·273	210·589	210·905	211·222	211·538
67	211·854	212·170	212·486	212·803	213·119	213·435	213·751	214·067	214·384	214·700
68	215·016	215·332	215·648	215·965	216·281	216·597	216·913	217·229	217·546	217·862
69	218·178	218·494	218·810	219·127	219·443	219·759	220·075	220·391	220·708	221·024
70	221·340	221·656	221·972	222·289	222·605	222·921	223·237	223·553	223·870	224·186
71	224·502	224·818	225·134	225·451	225·767	226·083	226·399	226·715	227·032	227·348
72	227·664	227·980	228·296	228·613	228·929	229·245	229·561	229·877	230·194	230·510
73	230·826	231·142	231·458	231·775	232·091	232·407	232·723	233·039	233·356	233·672
74	233·988	234·304	234·620	234·937	235·253	235·569	235·885	236·201	236·518	236·834
75	237·150	237·466	237·782	238·099	238·415	238·731	239·047	239·363	239·680	239·996
76	240·312	240·628	240·944	241·261	241·577	241·893	242·209	242·525	242·842	243·158
77	243·474	243·790	244·106	244·423	244·739	245·055	245·371	245·687	246·004	246·320
78	246·636	246·952	247·268	247·585	247·901	248·217	248·533	248·849	249·166	249·482
79	249·798	250·114	250·430	250·747	251·063	251·379	251·695	252·011	252·328	252·644
80	252·960	253·276	253·592	253·909	254·225	254·541	254·857	255·173	255·490	255·806

APPENDIX 22.

NOTES ON THE ARRANGEMENTS FOR AND MANAGEMENT OF LARGE WORKS.

(*Vide* Chapter II , paragraph 126 (*c*), page 175.)

INDEX TO PARAGRAPHS.

1 *Work Arrangements.*

2. *Miscellaneous.*

3. *Materials.*

I Work Arrangements.

1. General Arrangements for Works.—Before the works are commenced, a large scale plan should be made, and on this should be shown how the different offices, temporary works (such as kilns and mortar mills), in connection with manufacturing departments, and stores of materials are to be arranged most conveniently for the whole of the works. A liberal amount of space should be allotted to each.

The main office, the storeyard, workshop, and establishment quarters should be together in a central situation remote from the village, &c.

The material stacking-ground should be arranged so that the rough materials should be furthest from and the finished ones nearest to the works Thus for the preparation of mortar the order would be: Sand, kankar, fuel, kilns, and mortar mills. The stacking-ground should be divided into two main parts, one from which supplies are being drawn, and the other on to which they are being delivered The former should be quite cleared before fresh supplies are brought on to it.

All should be set out so as not to interfere with the works from their commencement up to their completion

2. Bench Marks.—These should be of a permanent nature and should be set up as frequently as possible, so that intermediate readings need never be necessary. A list of them should be kept by all levellers at the beginning of their note-books.

All permanent marks on the works should be cut reversed-V shape in section and leaded.

3. Setting-out Pillars.—At the ends of all principal lines of the works should be erected small masonry pillars, each capped by a slab, on which the centre line and the reduced level should be engraved so that the pillars may serve as permanent alignment and bench marks.

4. Setting-out Marks.—Distance marks should be fixed at right angles and opposite to the end of each chain at a specified distance from the centre line, and their chainage should be engraved on them. As far as possible all such marks should be fixed permanently and out of the way during construction. The toe-lines of all embankments and the centre lines of all works should be lockspitted before the works are begun.

5. Side-widths and Levels.—Tables of these for each part of the work should be prepared, and copies of them should be given to all who have to set out the work.

6. Foundation Plan.—A large scale plan of the foundations should be prepared, and all levels should be carefully recorded on it. It will be best to divide the area into small rectangular compartments, to take the mean level of each, and to note any sudden changes and the quantity of filling required to make up the work to the top or to any assumed level.

7. Foundation Courses.—Particular care is necessary to set these out to the correct widths and levels. On no account should one course be erroneously run into another: to avoid this on each course at intervals should be painted its reduced level. Teak battens should be fixed parallel to and near the face of the masonry works as profiles, and references to the proper position from them of the top edges of the courses should be marked on them.

8. Setting-out Platforms.—Archrings should be set out full size on plastered platforms, each voussoir being drawn thereon. Similarly, curved profiles, cut-water courses, caps, &c, should be drawn out full size.

It will assist the workmen if full-sized models, in mud masonry, &c., of difficult parts of the work are made.

9. Progress Sections.—A longitudinal section and cross-sections of progress effected should be maintained The top of the work as completed each month should be carefully levelled and plotted thereon, and each month's work should be indicated by distinctive colour. On the cross-sections levels should be recorded. Separate sections should be maintained for excavation and filling.

10. Programme of Work.—As soon as the works are fairly started, a programme of anticipated progress should be made. This should take into account the times by which certain stages must be completed, and must allow liberally for all contingencies. The actual progress of the work should be kept in advance of it, to allow for possible future delays.

11. Temporary Works.—These should invariably be set out neatly to proper lines and levels. Profiles of these works should be erected in advance whenever possible, so that their future positions and sizes may be seen at a glance.

12. Rapid Completion of Small Works.—Small works, repairs,

&c., specially ordered by the Officer, should be completed within three days of his order, so that further reference to them will be unnecessary. They should take precedence of the general routine work, and additional labourers (carpenter, blacksmith, &c) should be engaged, if necessary, to complete them.

13. Works Roads.—All roads should be set out outside the permanent works, and should be arranged to be carried up with them as they rise They should as a rule, be executed departmentally, and not by contractors No materials should be stacked on them If practicable, they should be arranged so as to be permanently useful after the completion of the works.

14. Tidying up Works.—The works must always be left in a tidy state at night, *e g* , all mortar should be used up or stacked , all concrete thoroughly rammed ; and watering cans, rammers, &c. returned to store. The Sub-Divisional Officer must himself see to this Rubbish, loose stones, &c , should not be allowed on the works or works roads; these can be cleared off by children

15. Water.—Arrangements should be made for water to be cheaply available at all parts of the work by means of pipes, pumps, and hose, cisterns, bhistees, &c It is essential that ample water is procurable throughout the fair season, and, if necessary, small storage reservoirs should be formed immediately at the end of the monsoon by damming nullas, &c.

16. Bailing.—For lifts up to 3 feet, and where there is plenty of room for working, the basket " sup " is the best means of bailing. It can also be used with double shifts up to 6 feet. Beyond the height, large or small pumps should be used. Bailing need not be done, as a rule, where the water level is below the bottom of the blast holes.

17. Reserve Work.—Certain work should be reserved for execution during rainy days or when the wetness of the ground will prevent the continuance of the main work Preparation of concrete metal is a good work for this purpose.

18. Preparation for the Monsoon.—All plant and material liable to be submerged should be removed to places of safety in ample time All burnt kankar should be slaked and the lime screened and stored All arrangements should be made beforehand for moving material to where it will be required, as this can be done most cheaply and cleanly in the fair weather.

19. Works Order-book.—A book should be opened in which all important orders are to be recorded. Each order should be numbered for reference and dated, and the orders should be separated into distinctive sections In a marginal column should be entered the date of completion of the order, and any additions to or modifications of it found necessary These latter should be made in red ink, so as to catch the eye A copy of the book should be kept by each Subordinate, who should send it daily to the office at 2 p.m. to have new entries made in it.

The orders given in this book should have all the force of those separately and specially communicated by letters, and they must be carried out as quickly as possible. Any information respecting the method of completion of an order, &c., which can be concisely given should be noted below the order.

20. Hand Sketches.—Dimensioned hand sketches should always be given to those directly in charge of any minor work, and these should be kept on it for ready reference.

21. Maistries' Order-book.—A rough note-book should be given to each Maistry, and in it extracts from the general works order-book which apply to his charge should be posted by the Subordinate in vernacular. Blank pages should be left for the Officer to write his orders on the spot ; these should be shown at once by the Maistry to the Subordinate, and should be translated by him. Important orders should be given in writing, not verbally, otherwise they are liable to misinterpretation. Verbal orders should not be accepted as authority for any change in the work

22. Maistries' Appliances.—Each Maistry should be provided with setting-out appliances, *i.e* , one 50-feet tape (common), mason's level, plumb-line, foot-rule, string, chalk, &c. A small boy may be told off to each Maistry to hold these, to run messages, &c. Muccadums may be given string, chalk, and pegs, &c.

23. Supervision of Construction.—Unless in exceptional circumstances all constructional work should be under the direct supervision of a Maistry, and should not be left to a Muccadum alone.

24. Boundaries of Acquired Land.—These should be permanently marked, and as soon as it is settled to acquire the land.

25. Survey Marks affected by Works.—A list of these should be sent at once to the Revenue authorities, with a view to the correction of their records, and their action in respect to the

removal and replacement of the survey marks awaited as long as possible.

Off-sets to fixed points should be taken from each, so that if their removal is urgent their original positions may be determined at any time.

26. Excavating for Materials in Land Temporarily acquired.—When land has to be returned to the owners, the top soil should be carefully removed and stacked separately, so that, after the required material (such as kankar) has been removed, this soil may be replaced so as to permit of the cultivation of the area. For the same reason the area should be left levelled, tidy and free from stones, rubbish &c.

27. Native Holidays.—One week's notice should be given to the Officer of all holidays which are likely to interfere with the progress of the work, so that due arrangements during them may be made in advance.

28. Works Postal Arrangements.—Each Subordinate and Maistry should be accompanied on the works by a small boy who can take messages In addition there should be a man who can take and deliver all official papers, &c , in a leather bag or tin case to all sections of the work.

2 MISCELLANEOUS.

29. Sanitation.—(*a*) No one should be allowed to go for relief or to wash clothes, bathe, eat, &c , above, or at, the source of water-supply.

(*b*) No one should resort for relief to within 100 feet of nulla and river beds.

(*c*) If the people will make use of trenches, these should be dug in neat lines and filled with earth each day, and fresh ones should be prepared in advance by the sweepers

(*d*) If the labourers will not use the trenches, specified squatting areas, some for men and others for women, should be marked out by yellow flags at reasonable and convenient distances from the work, where they are not likely to produce nuisance.

(*e*) Piecework gangs and all who camp on the works must keep clean their surroundings for 200 yards, or be made to live further off.

(*f*) Sanitary guards, with numbered yellow belts, should be appointed to see that these orders are carried out. Also, a few sweepers should in any case be engaged

(*g*) Specified watering-places, or wells, for each caste should be arranged for, it will save waste of time if cylinders of water are kept for them under guard on the works themselves. One or more buckets should be specially reserved for each watering-place, and others should not be used.

(*h*) A separate set of water guards, with numbered blue belts, should be appointed.

(*k*) A responsible karkun, &c., should be appointed to superintend the sanitary and water guards and he should report the names of all offenders against the rules.

30. Blasting.—Every precaution should be taken to avoid accidents.

(*a*) A responsible man should be put in charge.

(*b*) Blasting, as a rule, should take place at fixed times only, *e g.*, at the mid-day meal and at sunset.

(*c*) Only copper tamping bars and needles should be used, and they should be kept bright for the easy recognition of their material.

(*d*) A cordon of watchmen with red flags should be formed around the area of the blasts, and 300 feet from them. The flags should be shown ten minutes before blasting begins and until the last blast has gone off. In case all the blasts do not explode, the flags should not be lowered until ten minutes after the last explosion The holes of miss-fires should then be well wetted and their charges should be carefully removed after an interval of an hour.

(*e*) The number of blasts at one time and place should be limited to six, and these should be counted and recorded by the man-in-charge.

(*f*) Fresh blasting should not be carried out without the orders of the man-in-charge.

(*g*) Fuzes must be cut to the proper length *before* the holes are filled with powder and tamped.

31. Serious Accidents.—The sufferers should at once be taken to the works hospital, and the medical man summoned. The Officer and Subordinate should be informed as soon as possible.

3. Materials.

32. Material Requirements.—The quantities of materials required for a given amount of work should be estimated from time to time, so that stocks of different classes may be regulated to requirements, and, especially, when the work is approaching completion.

33. Materials at Site.—Sufficient material of all descriptions should be collected and kept available so as to last at least for one month, and preferably for two months This will prevent strikes for higher rates among suppliers and the stoppage of any work from deficiency of supply.

34. Collection of Materials.—A special man, not under the rank of Sub-overseer, should be told off to superintend the suppliers of materials, many of whom will be working some distance from the works. He should see that the material is up to specification and is being supplied according to ordered quantities He should separate rejected materials from the rest before measurements are taken, and should, as far as possible, prevent them from being brought to site. He should meet the Officer once every second day to keep him informed of what is being done. Lists of special requirements should be given to him from time to time

35. Selection of Materials.—A special and well-qualified man should be kept to see that all stones, metal, and other materials are of the specified size, quality, &c , before they are stacked for measurement Those which are considered to be not up to specification, should be marked at once with a distinctive temporary mark, stacked separately for inspection, and removed from the works, or marked permanently as soon as they have been finally rejected by the Officer or Subordinate.

36. Stacking of Materials.—Materials such as sand, slaked lime, kankar, charcoal and metal should be stacked under departmental supervision in heaps of specified height, so as to obtain fair measure. Rubble stones should be bought in heaps of specified dimensions, and a specified deduction made therefrom to allow for vacuities. The method of stacking should be carefully watched to prevent fraud

37. Issue of Materials.—No material should be issued to anyone except by the Maistry in charge, who should account for the issues to the Subordinate daily.

38. Balances of Materials at Site.—These should be actually measured by the close of the works month, as on them the proper calculation of the cost of the work depends. They should be reported on the second day of the new month by the Subordinate in charge of materials to the Officer. To facilitate this measurement, all paid-for heaps should have on top flat reference stones, with their

cubic capacity painted thereon. All these stones should be in place before the original measurements are taken. No heap should be broken into until the one last drawn upon is completely used up. The quantity of stock in hand can thus be readily ascertained at any time.

39. Protection of Materials.—Due precautions should invariably be taken to shelter all materials requiring protection from sun and rain, and to guard carefully all costly and readily saleable material. All wooden articles should be stacked on stones off the ground

40. Lime-burning Account.—Accounts of the cost of manufacture and the amount used on the works should be carefully kept. Also, the out-turn of each kiln should be recorded in a register

Where cement is used, a register of the issue of barrels should be kept, and the empty ones returned at once to store.

41 Gelatine and Dynamite, &c., Cartridges.—These cartridges should be kept in a separate locked under-ground magazine away from the works. Their detonators and fuze should be kept under lock in the store. The cartridges should be issued twice a day in time to allow of their being primed, and, until handed over to the blasters, must be in the charge of a responsible man. The junction of the fuze and the detonator should, when necessary, be made water-tight, with wax, tarred over. A careful register of issues, blasts, and returned cartridges should be kept. All unexploded cartridges should at once be returned to the magazine after the primers have been removed.

42. Powder.—Blasting powder should be stored in wooden casks buried under ground.

4. Stores and Tools.

43. Stores and Workshops.—A special sub-charge should be made of these. In regard to stores, the different sub-divisional Officers should frame monthly indents on the chief Storekeeper, so as to replenish their own stores, and these indents should be sanctioned by the head of the works.

Large repairs should be sent to the workshop, and small ones effected by the sub-divisional staff on the works themselves

44. The Store.—Each class of store should be kept separate from the rest, and all outside stores should be raised from the ground.

45. Stamping Tools.—All tools in use should be stamped. The

stamping of each letter should cost from two to three annas per hundred tools.

46. Register of Tools.—In this one or more pages should be devoted to each kind of tool in ordinary, general use. In it the dated issues (in black) to, and returns (in red) by, each petty Contractor and sub-division should be shown by the Storekeeper, so that he can readily ascertain and check his balance at any time.

47. Issue of Tools to Contractors.—A register of all issues and receipts should be kept, with a separate page, or pages, for each Contractor. Persons supplied should sign each entry, and afterwards the Subordinate should ascertain from them that they have received the full number entered against their names, and should also initial the entry. Each entry should give the number, size, weight, condition, &c., of the articles issued.

48. Issue of Plant to Contractors.—Where there are several petty Contractors on a work, a register of small plant, such as trucks, measures for concrete, &c., ladders and screens, &c., should be maintained, and the articles should be marked temporarily with the Contractor's initials.

49. Issue of Tools to Departmental Gangs.—Tools may similarly be issued to Muccadums of gangs, and may be kept with them provided that on the days on which musters are closed all such tools are counted by one of the staff. The value of missing tools should be recovered from the defaulters at the time of payment of the musters.

50. Returning Tools, &c., out of use.—All tools and articles of plant when no longer required on the work should at once be returned to store.

51. Broken Articles.—All such as cannot be mended on the works themselves should at once be sent to the workshop, and should be repaired immediately. If unrepairable, they should at once be weeded out and kept separately in the main store for final disposal.

52. Watering Pots.—Galvanized watering pots, being costly, should be issued only to special men, who should be held responsible for their damage or loss Common ones made of old kerozine tins should be generally employed. For masonry work the roses should be soldered on to prevent their loss. Roses are necessary only for watering metal to be mixed for concrete, and for concrete and masonry less than two days old; other watering should be done plentifully by buckets.

53. Rammers.—Wooden rammers should be made of "babhul" or other hard wood, 7 inches to 8 inches diameter at base, 5 inches to 6 inches diameter at top, and 8 inches high The handles of these and other tools are best made of "babhul," or other tough wood; their ends should be split, and they should be wedged up tight.

54. Miscellaneous Articles.—A sufficient stock of all miscellaneous articles required on the works, *e.g*, mortar mill poles and centre posts, mortar millstones, scrapers, roller poles, ladders, boning rods, handles, rammers, wooden lime rakes, templates, tramway fittings, lime, cement, and metal measures, &c, should always be kept ready A list of the stock required should be made out and kept for guidance in the workshop.

A list of fittings and bolts, showing the amount of stock required to be maintained, balance on works and supplies to be procured, should be submitted monthly As soon as any stock article is issued another should be made to replace it.

Similarly, a month's supply of all petty stores should be kept by the Storekeeper, and should be replenished by means of monthly indents on the dealers.

55. Setting-out Materials.—These should invariably be kept in readiness in the sub-divisional store. They comprise teak templates, bamboos, pegs (large and small), string, coir, rope, chalk, white, black and red paint, tar, brushes, plumb-bobs, mason's squares, mason's levels, &c.

56. Indent for Following Day's Requirements.—Each Subordinate should prepare this by 2 p m., and get the Officer's sanction, so that issues may be made from the store in time Dimensions of articles and sketches should be given when necessary, and the indent should be arranged under heads in accordance with the separate minor works for which the articles are required, so that their costs may be charged to those works.

57. Register of Expenditure of "Sundries."—This should be kept regularly and posted daily, each entry being initialled by the Officer in charge to see that expenditure is properly controlled and debited to the works concerned.

58. Receipts and Issues.—All articles and materials sent from one part of the works to another should be accompanied by vouchers or acknowledgments, which should be obtained and recorded so as to prevent mistakes occurring in debiting the charges incorrectly.

5 Plant.

59. Rules to be Observed in the Use of Tram Plant.—(*a*) Immediately a truck is in need of repair it should be taken off the rails and placed neatly on one side until it is repaired

(*b*) Truck men should not run with their trucks, but should proceed quietly.

(*c*) Trucks should not be loaded above their tops.

(*d*) If a loaded truck leaves the rails, it should be emptied before it is *lifted* on Bars should never be used for levering trucks on to the rails.

(*e*) All axles should be regularly oiled and a piece of oiled waste kept in each pedestal.

(*f*) The tram line should be laid in regular lines and curves, with as few ups and downs as possible It should be maintained well ballasted, and all ties and joints, &c., should constantly be kept in repair. All switches and points should be kept free of sand, dirt, &c. Where field drainage is crossed, the line should be laid on single stone sleeper supports.

(*g*) All trucks and carriages should be numbered. They should be examined each evening in the presence of their users near the sub-divisional office ; all missing parts should be noted, and they should be replaced at the cost of the users The moving parts of the carriages should then be oiled ready for next day's work

(*h*) All trucks should be examined once a week by the Sub-divisional Officer himself

(*i*) All repairs should be done by skilled blacksmiths

(*j*) All loose parts of the tramway and trucks should be neatly stacked near the workshop or store

(*k*) The number of serviceable trucks should be noted in the weekly report.

(*l*) A special gang should be kept for tram laying, &c. When the line has once been well laid, a very small number of men will suffice for this purpose.

(*m*) A standard gauge for rail laying should be made and given to the rail layers.

(*n*) To avoid confusion, trucks should go in "trains", *i.e.*, a number together.

(*o*) As a rule trams should not be used at night, nor when there is not departmental supervision.

60. Supervision of Pumping and other Engines.—These must invariably be placed under the charge of a thoroughly competent and certificated man He should be supplied with the best materials and appliances, including special dubbin for belts. He should be held solely responsible for everything under his charge. All engines should be thoroughly cleaned up, washed out and repaired once a week

61. Pumping Engines and Special Plant.—These should be carefully and thoroughly inspected once a week by the Subordinate in charge of the workshop Petty repairs should be done by skilled men only, and all materials used should be of the best quality. Such plant should be protected from the weather, and should be cleaned and painted before the rains.

62. Pumps.—All valve leathers should continuously be kept soft. All valves should be maintained in a state of repair. Hose should be repaired as soon as this becomes necessary; should be carefully coiled when out of use; and should be lifted, not dragged, over rough ground. Suction hose should be wrapped round with thin coir rope to protect it from abrasion and the action of light.

63. Boat.—This should be kept under cover as much as possible, and should be loaded when it is out of use, so as to keep under water all planks liable to be submerged when it is in use, and thus to prevent their cracking The lower sides of gratings should be tarred and the upper sides oiled All unpainted wood should be oiled and brass work kept bright. The leather buttons on the oars should be greased with fat, and the oars stacked on level planks to prevent them warping The inside of the boat should be kept dry. Coir rope fenders should be provided to protect the sides of the boat from injury.

In reservoirs, boats should be anchored thus: An old millstone, or other heavy weight, should be placed on the reservoir bed, and to it should be fastened a chain which is attached at the other end to a buoy, to which the boat itself should be fastened.

64. Stacking Woodwork.—All wood and wooden articles out of use should be neatly stacked under shade and raised from the ground on stones

65. Removal of Plant and Material before Monsoon.—All plant and material liable to be submerged, or to be carried away, should be removed to places of safety by May 15th. No fresh material should thereafter be stacked where it is liable to suffer damage.

6 Establishment

66. Sub-division of Works.—On a large work each Upper Subordinate should have a definite charge, and should have special establishment and a small store allotted to him.

67. Duties of Establishment.—The duties of each member should be laid down in writing, and each should be held wholly and solely responsible for his definite charge.

68. Appointment of Work Establishment.—This should be done according to some scale, thus :—

1 Maistry to 50 Masons ;
1 Karkun to 200 labourers ;
1 Muccadum to 50 labourers.

These are in addition to the general supervising staff.

69. Appointments on Works.—All appointments should be made by the Officer of highest rank on the work, or be sanctioned in writing by him Also, all changes in pay and classification should similarly be sanctioned.

70. Muccadums' Pay.—The pay of these men should bear some relation to the number of people they bring on to the works. They should be held responsible for keeping up the strength of their gangs and for giving early information about those who intend to leave or have left the works

71. Muccadums' Badges.—These should consist of red cloth, 3 inches in diameter, on which distinctive numbers, about 2 inches high, should be printed in black, so that the men can be identified at any time. The badges should be sewn on to their left sleeves.

72. Independence of Maistries and Karkuns.—Maistries should not have control over Karkuns, and *vice versâ*. Each should be quite independent of the other.

73. Authority of Office Establishment on Works.—Clerks and peons should not have authority on works, and coolies and materials should not be given to them without written orders.

74. Daily Inspections.—The Maistries should accompany the Officer when he commences his inspection, and the Subordinate, on his return journey All orders for work and complaints should be disposed of by the Officer with the latter at the time, and labourers should thus be informed.

7. Labour.

75. Working Hours.—The commencement and end of the working day should be announced by the blowing of a horn Half an hour after sunrise and sunset itself may be fixed for these limits. Mustering should begin at once, and late arrivals should be kept separate, so they may be dealt with afterwards.

76. Off Time.—Specified times should be fixed for meals, and no one should be allowed to eat at other times. The commencement and end of "off time" should be announced by a horn, and a flag should be kept hoisted in a conspicuous place during its continuance. In cases where rapid progress is necessary, continuous work should be arranged for by employing special gangs when the ordinary ones are off the work

77. Rates and Numbers of Labourers.—A scale of the maximum and the minimum rates of labour should be laid down Thereafter the Subordinate in charge may be authorised to fix the rates in individual cases. Similarly, he should fix the number required for any particular work.

Although Maistries and Karkuns may be consulted in such matters, they must not be allowed to fix rates and settle numbers.

78. Tasking Work.—Rates should be fixed for all classes of work, and the labourers kept up to their allotted tasks and paid accordingly. Due notice of their work should be given to them and its extent clearly explained to them. For certain classes of work the ticket system may be used, but quantities of work done should be checked by tape measurements periodically.

79. Distribution of Work.—By 5 p m every one should know what and where is to be his work for the following day, so that then he may at once proceed to it. The Officer should give his orders to the Subordinates, these to the Maistries, and these to the gang Muccadums

8. Petty Contractors.

80. Agreements with Petty Contractors and Suppliers.—These should be recorded in a book, each being signed or attested by the supplier, &c. They should state shortly :—

(*a*) The nature, specification, and quantity of supply, &c.

(*b*) The rate ;

(*c*) The time by which quantities of supply, &c., are to be effected and measurements to be taken ;

(*d*) The penalties to be enforced for late, short, or inferior supplies, and for inattention to orders.

It is best to have agreements with single men, and not with partners. All agreements should be subject to the approval of the final sanctioning authority

81. Petty Supply Contract.—A contract to last for one year should be entered into with a respectable dealer to supply all miscellaneous articles required on the works. Monthly indents prepared by the Storekeeper on receipt of the sub-divisional indents should be sent to him, and he should be bound to supply all articles within a fixed time.

82. Rates of Supply by Petty Contractors.—As soon as rates are settled, a list of them should be made out and posted, so that all intending suppliers may know them. More liberal rates may be given at first to attract labour and material, and the dates up to which they will be in force should be intimated. After fair rates have once been settled, it is not advisable to make further alterations in them. Where the same materials have to be procured from different localities, a table of additional rates for carriage should be made out.

83. Supervision of Petty Contractors' Work, &c.—Reliable Muccadums should be told off to see that all such works are properly carried out.

Similarly, special men should be told off to superintend every departmental or other operation, such as lime burning, sand washing, mortar mixing, &c

84. Number of Petty Contractors' Labourers.—Each Maistry should keep a tabular register of the numbers of each labourers on each work under him, so that in case of dispute about the fairness of rates the matter may be inquired into The Subordinate should check these and abstract them daily in a book or register.

85. Fining Contractors' Workpeople.—A list of fines should be prepared monthly and the total recovered from the Contractor, either by deduction from his bill or by ready money payment. The list should be sanctioned by the Officer in charge.

9. Office Arrangements.

86. Sub-Divisional Office Duties.—The Sub-divisional Clerk should audit and check daily reports, muster rolls, bills, measurement

books, and other cash vouchers and imprest accounts. He should prepare daily reports and other returns, day books, registers of works, and all account papers and returns exclusive of those given to the Cashier

The Second Clerk should register, copy, despatch, and file all correspondence.

The Cashier should complete measurement books, prepare receipts, bills, and imprest accounts, and compare correspondence. He should bring cash from the treasury, effect all payments, and be responsible for all cash and cash transactions. He should initial all cheques in token of having examined and found them correct. When making payments of muster rolls, he should count out only a little more money than is required and, when the payment is finished, he should check the balance with the rolls before returning it to the cash bags.

The Storekeeper should prepare store accounts and indents for store supplies. He should maintain careful registers of all tools and plant and store issues. He should keep a sufficient balance of everything required on the works, and make timely indents for all supplies of which the store is short.

87. Sub-divisional Returns.—These should be posted directly the entries can be made, so as to avoid a rush of work at the end of the month.

88. Final Measurements.—Before these are made, all side widths and slopes should be properly taken out, and, before the bills are prepared all " deadmen " should be removed and the foundation cleared Cross and longitudinal sections of completed foundations should be taken as soon as they are ready, and the length to which the cross-section applies should be noted carefully.

89. Bills and Measurement Books.—All persons connected with the preparation or audit of these should enter their dated initials to the transaction concerned.

90. Daily Measurements of Work.—These are to be taken each evening with such accuracy that their total will not differ more than 5 per cent from the month's final measurements.

91. Final Measurements of Work and Supplies.—These should be recorded in measurement books, each chain of work being separately entered and its totals carried forward, so that a comparison and a check with the estimate can be made at any time.

Similarly, the measurements of each class of material should be separately recorded

92. Daily Report.—This should be submitted each day before 2 p m on a printed form having columns for :—

The numbers on each subwork of each principal class of labour, one being for establishment ;

The approximate quantities of work done during the previous day ;

Remarks about any particular circumstance, reasons for increase or decrease of labourers, or out-turn, &c

93. Weekly Report.—The daily reports should be abstracted into a printed weekly report, which should record the daily numbers of different classes of labourers and the approximate total amount of work done. On the back should be entered general remarks noting :—

The chainage and levels to which the work was completed ;

The quantity of work completed ;

Reasons for slow or quick progress ,

Any particular occurrences during the week.

The weekly reports should thus give a complete and fairly accurate history of the works.

APPENDIX 23.

TABLE[1] OF TASK-WORK IN DIFFERENT SOILS.

Explanatory Note.

The table has been drawn up for the purpose of determining and arranging in a systematic manner the quantities of work required as daily tasks from men, women, and children engaged upon the construction of earthwork, and has proved particularly useful where large bodies of workpeople, such as famine or extra-mural labour, have been employed.

The workpeople should be divided into gangs, each gang consisting of fifty people—men, women, and children—under one muccadum; six gangs being placed under one mustering karkun.

The following example illustrates the use of the Table :—

Assume a gang consisting of 20 men + 20 women + 10 children to be working with a lift varying from 6 to 10 feet, and a lead varying from 200 to 300 feet, in soft muram.

In the column including this lift and lead, the relative tasks for one man, one woman, and one child are 48, 32, and 16 cubic feet respectively :—

$$\begin{aligned} \text{Then } 48 \times 20 &= 960 = \text{task of 20 men.} \\ 32 \times 20 &= 640 = \text{,, ,, 20 women.} \\ 16 \times 10 &= 160 = \text{,, ,, 10 children} \\ &\overline{1{,}760} \text{ cubic feet} = \text{total task} \end{aligned}$$

to be executed by the gang, if the material were black or red soil. As, however, the example refers to soft muram, the multiplier, 0·82 (see the footnote to the Table) must be used :

$$\text{Total task} = 1{,}760 \times 0{\cdot}82 = 1{,}443{\cdot}20 \text{ cubic feet,}$$

and this is given to the muccadum in round numbers Thus, in the case of a puddle trench 10 feet in width the task would be given as follows :—

Length	Width.	Depth
140 feet ×	10 feet ×	1 foot.

[1] Extracted from Marryat's "Specifications, Rates and Notes on Work," 4th edn pp. 444–448. (Drawn up by the late Mr. C. T. Burke, B.E., M.Inst.C.E.)

In the construction of a dam where the earth is put on in layers of uniform thickness, tasks should be measured on the dam

For breaking lumps of earth, spreading and making the layers uniform, two men should be allowed for each gang—that is, one man for each twenty-five people employed

In puddling, in layers of 1 foot in thickness, three men should complete, including cutting, treading and covering, a space of 25 feet × 10 feet, or 250 cubic feet in one day.

In "ramming" with a rammer weighing 30 lbs, one man should complete 500 square feet per day.

The following Tables have reference more especially to earthwork in large dams and embankments :—

TABLE SHOWING AN AVERAGE DAY'S WORK PER MAN, WOMAN, AND CHILD UNDER DIFFERENT "LIFTS" AND "LEADS" IN BLACK OR RED SOIL

Lift, feet.		Lead, feet		Average Day's Work, cubic feet		
From	To	From	To	Man	Woman	Child
..	5 inclusive.	..	50	76	51	25
		50	100	70	47	23
		100	200	64	43	21
		200	300	59	39	20
		300	400	53	35	18
		400	500	48	32	16
		500	600	42	28	14
		600	700	36	24	12
		700	800	30	20	10
		800	900	24	16	8
		900	1,000	18	12	6
5	10 inclusive.	..	100	56	37	19
		100	200	52	35	17
		200	300	48	32	16
		300	400	44	29	15
		400	500	40	27	13
		500	600	35	23	12
		600	700	31	21	10
		700	800	27	18	9
		800	900	22	15	7
		900	1,000	17	11	6

TABLE—*continued.*

Lift, feet		Lead, feet		Average Day's Work, cubic feet		
From	To	From	To	Man	Woman	Child
10	15	..	100	48	32	16
inclusive.		100	200	45	30	15
		200	300	41	27	14
		300	400	38	25	13
		400	500	34	23	11
		500	600	30	20	10
		600	700	27	18	9
		700	800	24	16	8
		800	900	20	13	7
		900	1,000	16	11	5
15	20	..	100	42	28	14
inclusive.		100	200	39	26	13
		200	300	36	24	12
		300	400	33	22	11
		400	500	30	20	10
		500	600	27	18	9
		600	700	24	16	8
		700	800	21	14	7
		800	900	18	12	6
		900	1,000	15	10	5
20	25	..	100	37	25	12
inclusive.		100	200	35	23	12
		200	300	32	21	11
		300	400	30	20	10
		400	500	27	18	9
		500	600	25	17	8
		600	700	22	15	7
		700	800	19	13	6
		800	900	17	11	6
		900	1,000	14	9	5
25	30	..	100	33	22	11
inclusive.		100	200	31	21	10
		200	300	29	19	10
		300	400	27	18	9
		400	500	25	17	8
		500	600	23	15	8
		600	700	21	14	7
		700	800	19	13	6
		800	900	16	11	5
		900	1,000	13	9	4

TABLE—*continued.*

Lift, feet.		Lead, feet.		Average Day's Work, cubic feet.		
From	To	From	To	Man	Woman	Child.
30	35	—	100	30	20	10
inclusive.		100	200	28	19	9
		200	300	26	17	9
		300	400	24	16	8
		400	500	22	15	7
		500	600	20	13	7
		600	700	18	12	6
		700	800	16	11	5
		800	900	14	9	5
		900	1,000	12	8	4
35	40	—	100	28	19	9
inclusive.		100	200	26	17	9
		200	300	24	16	8
		300	400	22	15	7
		400	500	20	13	7
		500	600	18	12	6
		600	700	16	11	5
		700	800	14	9	5
		800	900	13	9	4
		900	1,000	12	8	4
40	45	—	100	26	17	9
inclusive.		100	200	24	16	8
		200	300	23	15	8
		300	400	21	14	7
		400	500	20	13	7
		500	600	18	12	6
		600	700	17	11	6
		700	800	15	10	5
		800	900	13	9	4
		900	1,000	11	7	4
45	50	—	100	24	16	8
inclusive.		100	200	23	15	8
		200	300	21	14	7
		300	400	20	13	7
		400	500	18	12	6
		500	600	17	11	6
		600	700	15	10	5
		700	800	14	9	5
		800	900	12	8	4
		900	1,000	10	7	3

TABLE—*continued.*

Lift, feet.		Lead, feet.		Average Day's Work, cubic feet		
From	To	From	To	Man	Woman.	Child
50 inclusive.	55	—	100	22	15	7
		100	200	21	14	7
		200	300	20	13	7
		300	400	18	12	6
		400	500	16	11	5
		500	600	15	10	5
		600	700	14	9	5
		700	800	13	9	4
		800	900	12	8	4
		900	1,000	10	7	3
55 inclusive.	60	—	100	21	14	7
		100	200	20	13	7
		200	300	18	12	6
		300	400	17	11	6
		400	500	15	10	5
		500	600	14	9	5
		600	700	13	9	4
		700	800	12	8	4
		800	900	10	7	3
		900	1,000	9	6	3

The above quantities must be multiplied for :—

Soft muram by	0·82	Excavation.
Average muram by . . .	0·64	Excavation.
Hard muram by	0·46	Excavation.
Loose earth or sand by . . .	1·25	Spoil.
Loose muram by	1·00	Spoil.

APPENDIX 24.

TABLES[1] FOR ASCERTAINING THE COST OF CARRIAGE BY COOLIES AND BY CARTS.

Note on the Cost of Carriage.

When any quantity of materials of given weight or cubic content (L) has to be moved to any distance in feet (d) by a succession of trips of a number of units of conveyance, whose loads in lbs or cubic feet is (l), then the cost of transport of (L) will depend :—

Firstly, on X, or the proportion of (L) to (l), that is, the number of unit-loads, or trips of a unit of conveyance, that will be necessary.

Secondly, on Y, or the fraction of a working day, taken by a unit to make one trip to (d) and back.

Thirdly, on Z, or the hire of a unit of conveyance for a working day.

In fact, the cost of transport will $= X \times Y \times Z$

Tables for all possible values of Z would have to be very numerous The following Tables, however, give some useful values of Y for various leads, also of $X \times Y$ for a few given values of X or $\frac{L}{l}$.

The calculation of Y requires the following data .—

M, the number of minutes in a working day

t, the time in minutes taken to load and unload a unit

*s, the average speed in feet per minute of a unit

If, also, for convenience of notation,

T, represent the time in minutes taken for a unit to make one trip to (d) and back,

N, be the number of such trips made in a working day,

then

$$T = \frac{2d}{s} + t \quad . \quad . \quad . \quad . \quad (1)$$

$$N = \frac{M}{T} = \frac{M}{\frac{2d}{s} + t} \quad . \quad . \quad . \quad . \quad (2)$$

$$Y = \frac{T}{M} = \frac{\frac{2d}{s} + t}{M} = \frac{1}{N} \quad . \quad . \quad . \quad (3)$$

If T be observed as well as t, then from (1) .—

$$s = \frac{2d}{T - t} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

* Note —s, the average speed, is the harmonic mean between the observed or known speeds s', s'', of the unit when loaded and unloaded respectively,

i e $s = \frac{2\,s'\,s''}{s' + s''}$

[1] Extracted from Marryat's "Specifications, Rates, and Notes on Work," 5th edn., pp. 2–4.

TABLE I.

When the unit of conveyance = a COOLY LOAD, or $l = \frac{1}{2}$ cubic foot.
DATA: $M = 500$ minutes; $t = \frac{1}{2}$ minute, $s = 200$ feet; d as per column I.

Formulæ $T = \frac{2d}{s} + t$, $N = \frac{M}{T}$, $Y = \frac{T}{M}$; $X = \frac{L}{l}$; Z = daily hire of unit

Cost of transport of quantity L to $d = X \times Y \times Z$.

I	II	III	IV	V	I	II	III.	IV	V
Lead or distance in feet	No. of trips to d and back in a day.	Time of a trip to d and back in minutes	Fraction of a day taken to make one trip to d and back expressed in decimals	When $l = \frac{1}{2}$ cft and L = 100 cubic ft, X × Y = 200 Y	Lead or distance in feet.	No of trips to d and back in a day	Time of a trip to d and back in minutes	Fraction of a day taken to make one trip to d and back expressed in decimals	When $l = \frac{1}{2}$ cft and L = 100 cubic ft, X × Y = 200 Y
d	N	T	Y	X × Y	d.	N.	T	Y	X × Y
20	714	0·70	·0014	0 28	475	95	5 25	0105	2·10
30	625	0 80	0015	0 32	500	91	5·50	0110	2 20
40	556	0 90	0018	0 36	525	87	5 75	0115	2 30
50	500	1·00	0020	0 40	550	83	6 00	0120	2·40
60	455	1·10	·0022	0 44	575	80	6 25	0125	2 50
70	417	1 20	·0024	0 48	600	77	6 50	0130	2 60
80	385	1·30	0026	0 52	625	74	6 75	0135	2·70
90	357	1 40	·0028	0 56	650	71	7 00	0140	2 80
100	333	1·50	0030	0 60	675	68	7·25	·0145	2·90
125	286	1 75	·0035	0 70	700	65	7 50	·0150	3·00
150	250	2·00	·0040	0 80	750	62	8·00	0160	3 20
175	222	2 25	·0045	0 90	800	59	8 50	·0170	3·40
200	200	2 50	0050	1 00	850	56	9 00	·0180	3 60
225	182	2·75	·0055	1 10	900	53	9 50	·0190	3 80
250	167	3 00	·0060	1·20	950	50	10 00	0200	4·00
275	154	3·25	0065	1 30	1,000	48	10·50	0210	4·20
300	143	3 50	·0070	1 40	1,050	45	11 00	0220	4 40
325	133	3 75	·0075	1·50	1,100	43	11·50	·0230	4·60
350	125	4 00	·0080	1 60	1,150	42	12·00	·0240	4 80
375	118	4 25	0085	1 70	1,200	40	12·50	0250	5 00
400	111	4·50	·0090	1·80	1,250	38	13 00	0260	5 20
425	105	4 75	·0095	1 90	1,300	37	13 50	0270	5·40
450	100	5 00	0100	2 00	1,320	36 5	13 75	·0275	5 50

N B.—Quantity carried in a day is N unit-loads

Cost of carrying one cooly load to d = Y × hire of cooly;

Cost of carrying 100 cubic feet of any materials is X × Y × hire of cooly.

EXAMPLE.

What would be the cost of removing 100 cubic feet to 600 feet if coolies cost 3 annas a day?—Ans. 2·6 × 3 = 7·8 annas.

APPENDIX 24.

TABLE II.

When the unit of conveyance = a CART LOAD, or l has the values in Columns V. to XII.

DATA. M = 540 minutes; $t = 10$ minutes; $s = 100$ feet; d as per column I.

Formulæ :—$T = \frac{2d}{s} + t$, $N = \frac{M}{T}$; $Y = \frac{T}{M}$, $X = \frac{L}{l}$; Z = daily hire of unit.

Cost of transport of quantity L to d = X × Y × Z.

I.	II	III.	IV.	V	VI	VII	VIII	IX	X.	XI.	XII
Lead or distance in feet.	No of trips to d and back in a day	Time of a trip to d and back in minutes	Fraction of a day taken to make one trip to d and back expressed in decimals	X × Y FOR THE UNDERMENTIONED VALUES OF L AND l.							
				L = 100 cubic feet					L = 1 ton		
				l in cft	l in cft	l in cft	l in cft	l in cft.	l in cwt	l in cwt	l in cwt
d	N	T.	Y.	6	7	8	9	10	8	9	10
400	30	18·0	0·0333	0·556	0·476	0·416	0·370	0·333	0·0825	0·0730	0·0666
431	29	18·6	0·0345	0·575	0·493	0·431	0·383	0·345	0·0863	0·0766	0·0690
482	28	19·6	0·0357	0·595	0·510	0·446	0·397	0·357	0·0895	0·0793	0·0714
500	27	20·0	0·0370	0·617	0·529	0·463	0·411	0·370	0·0925	0·0826	0·0740
540	26	20·8	0·0384	0·640	0·549	0·480	0·427	0·384	0·0960	0·0853	0·0768
580	25	21·6	0·0400	0·666	0·572	0·500	0·444	0·400	0·1000	0·0889	0·0800
625	24	22·5	0·0416	0·693	0·594	0·520	0·462	0·416	0·1040	0·0925	0·0832
675	23	23·5	0·0435	0·724	0·621	0·544	0·483	0·435	0·1087	0·0966	0·0870
730	22	24·5	0·0454	0·757	0·649	0·567	0·504	0·454	0·1135	0·1009	0·0908
785	21	25·7	0·0476	0·793	0·680	0·595	0·529	0·476	0·1202	0·1068	0·0952
850	20	27·0	0·0500	0·833	0·714	0·625	0·555	0·500	0·1250	0·1111	0·1000
921	19	28·4	0·0526	0·877	0·751	0·657	0·584	0·526	0·1315	0·1169	0·1052
1,000	18	30·0	0·0555	0·925	0·793	0·694	0·617	0·555	0·1389	0·1234	0·1111
1,088	17	31·8	0·0588	0·980	0·840	0·735	0·653	0·588	0·1470	0·1307	0·1176
1,188	16	33·7	0·0625	1·042	0·893	0·781	0·694	0·625	0·1560	0·1390	0·1250
1,300	15	36·0	0·0666	1·111	0·952	0·833	0·740	0·666	0·1666	0·1480	0·1330
1,428	14	38·5	0·0714	1·190	1·020	0·893	0·793	0·714	0·1787	0·1590	0·1430
1,575	13	41·5	0·0769	1·282	1·099	0·961	0·854	0·769	0·1925	0·1710	0·1540
1,750	12	45·0	0·0833	1·389	1·190	1·042	0·926	0·833	0·2080	0·1850	0·1666
1,955	11	49·0	0·0909	1·515	1·299	1·136	1·010	0·909	0·2280	0·2020	0·1820
2,200	10	54·0	0·1000	1·666	1·429	1·250	1·111	1·000	0·2500	0·2220	0·2000
2,500	9·0	60·0	0·1111	1·852	1·587	1·389	1·234	1·111	0·2770	0·2470	0·2220
2,675	8·5	63·5	0·1176	1·960	1·680	1·470	1·307	1·176	0·2940	0·2610	0·2360
2,875	8·0	67·5	0·1250	2·083	1·786	1·563	1·389	1·250	0·3130	0·2770	0·2500
3,100	7·5	72·0	0·1333	2·222	1·904	1·666	1·481	1·333	0·3330	0·2960	0·2660
3,355	7·0	77·0	0·1428	2·383	2·040	1·784	1·587	1·428	0·3560	0·3170	0·2860
3,653	6·5	83·0	0·1538	2·563	2·197	1·922	1·709	1·538	0·3840	0·3410	0·3070
4,000	6·0	90·0	0·1666	2·777	2·381	2·083	1·852	1·666	0·4170	0·3700	0·3330
4,425	5·5	98·5	0·1818	3·030	2·597	2·273	2·020	1·818	0·4540	0·4030	0·3630
4,900	5·0	108·0	0·2000	3·333	2·857	2·500	2·222	2·000	0·5000	0·4450	0·4000
5,500	4·5	120·0	0·2222	3·703	3·174	2·777	2·409	2·222	0·5550	0·4940	0·4440
6,200	4·0	135·0	0·2500	4·166	3·571	3·125	2·777	2·500	0·6250	0·5560	0·5000
7,200	3·5	154·0	0·2857	4·762	4·082	3·573	3·174	2·857	0·7140	0·6340	0·5710
8,500	3·0	180·0	0·3333	5·555	4·762	4·167	3·704	3·333	1·8330	0·7410	0·6600
10,300	2·5	216·0	0·4000	6·666	5·714	5·000	4·444	4·000	1·000	0·8890	0·8000
13,000	2·0	270·0	0·5000	8·333	7·143	6·250	5·555	5·000	1·250	1·111	1·000
17,500	1·5	360·0	0·6666	11·111	9·524	8·333	7·407	6·666	1·670	1·480	1·333
26,500	1·0	540·0	1·000	16·666	14·286	12·500	11·111	10·00	2·500	2·220	2·000
53·500	0·5	1,080·0	2·000	33·333	28·572	25·000	22·222	20·00	5·000	4·440	4·000

EXAMPLE

The cost of carrying 100 cubic feet of metal 2,200 feet, if a cart hired at 12 annas a day takes 8 cubic feet as its load,

= 1·25 × hire of cart per day = 1·25 × 12 = 15 annas.

APPENDIX 25.

USEFUL MEMORANDA.[1]

I. General.

1 cubic foot of water = 62·425 lbs. = 0·557 cwt = 0·028 ton
1 cubic inch of water = 0·03612 lb.
1 gallon of water = 10 lbs. = 0·16 cubic foot.
1 cubic foot of water = 6·24 gallons = say, $6\frac{1}{4}$ gallons
1 cwt. of water = 1·8 cubic foot = 11·2 gallons
1 ton of water = 35·98 cubic feet = 224 gallons.
Inches of rainfall × 2,323,200 = cubic feet per square mile.
Inches of rainfall × $14\frac{1}{2}$ = millions of gallons per square mile.
Inches of rainfall × 3,630 = cubic feet per acre
1 inch run-off per hour per square mile = 645·33 cubic feet per second;
= say, one cubic foot per second per acre.
12 cubic feet per second = 1,036,800 cubic feet per day,
= say, 1 million cubic feet per day.
1 cubic foot per second = 31,536,000 cubic feet per year
1 cubic foot per minute = 9,000 gallons a day.
1 cubic foot per second = 540,000 gallons a day.
= say, half a million gallons a day.
Number of seconds in 1 day = 86,400.
1 horse-power = 33,000 foot lbs. per min. = 550 foot lbs. per sec

$$\text{Horse-power} = 62{\cdot}4 \times \frac{\text{cubic feet falling per second}}{550} \times \left(\begin{array}{c}\text{height of fall}\\ \text{in feet}\end{array}\right)$$

1 horse-power = approximately 8·814 cubic feet per second falling 1 foot.
Square feet in 1 acre = 43,560
Square feet in 1 square mile = 27,878,400
Acres in 1 square mile = 640.
1 acre foot = quantity of water 1 foot deep on 1 acre = 43,560 cubic feet = say, 1 cubic foot per second flowing for 12 hours.
1 million cubic feet = 22·9568, say, 23 acre feet.
Feet per second × 0·68 = miles per hour.

[1] Extracted chiefly from Molesworth's "Pocket Book of Engineering Formulæ," and Marryat's "Specifications, Rates, and Notes on Work."

I[A]. Table of the Discharge of the Run-off at one Inch an Hour from Catchment Areas, in Cubic Feet per Second

Cubic Feet per Second

Square miles	0	1	2	3	4	5	6	7	8	9
0	—	645·33	1,290·67	1,936 00	2,581·33	3,226·67	3,872 00	4,517 33	5,162 67	5,808 00
10	6,453	7,099	7,744	8,389	9,035	9,680	10,325	10,971	11,616	12,261
20	12,907	13,552	14,197	14,843	15,488	16,133	16,779	17,424	18,069	18,715
30	19,360	20,005	20,651	21,296	21,941	22,587	23,232	23,877	24,523	25,168
40	25,813	26,459	27,104	27,749	28,395	29,040	29,685	20,331	30,976	31,621
50	32,267	32,912	33,557	34,203	34,848	35,493	36,139	36,784	37,429	38,075
60	38,720	39,365	40,011	40,656	41,301	41,947	42,592	43,237	43,883	44,528
70	45,173	45,819	46,464	47,109	47,755	48,400	49,945	49,691	50,336	50,981
80	51,627	52,272	52,917	53,562	54,208	54,853	55,499	56,144	56,789	57,435
90	58,080	58,725	59,371	60,016	60,661	61,307	61,952	62,597	63,243	63,888
100	54,533	—	—	—	—	—	—	—	—	—

I[B] Table of the Yield of the Run-off of One Inch from Catchment Areas, in Million Cubic Feet

Million Cubic Feet

Square miles	0	1	2	3	4	5	6	7	8	9
0	—	2·323	4·646	6 970	9 293	11·616	13 939	16 262	18 586	20 909
10	23 232	25·555	27·878	30 202	32·525	34 848	37·171	39 494	41·818	44·141
20	46·464	48·787	51·110	53 434	55 757	58 080	60 403	62 726	65 050	67 373
30	69 696	72·019	74·342	76·666	78 989	81 312	83 635	85 958	88 282	90 605
40	92 928	95 251	97·574	99·898	102 221	104·544	106 867	109 190	111 514	113·837
50	116·160	118 483	120 806	123 130	125·453	127 776	130 099	132 422	134 746	137·069
60	139 392	141 715	144·038	146 362	148·685	151 008	153 331	155 654	157·978	160·301
70	162 624	164·947	157·270	169·594	171·917	174 240	176 563	178 886	181 210	183·533
80	185 856	188 179	190·502	192 826	195 149	197 472	199·795	202 118	204 442	206·765
90	209·088	211·411	213·734	216·058	218 381	220·704	223 027	225 350	227·674	229·997
100	232 320	—	—	—	—	—	—	—	—	—

II. Pressure of Water.

P = Pressure in lbs per square inch

H = Head of water in feet.

P = H × 0·4335 H = P × 2·307.

Pressure in lbs per square foot = 62·4 H.

Pressure of Water at Different Heads.

H = Head in feet.

P = Pressure in cwts. per square foot = 0·5574 H.

p = pressure in lbs per square inch = 0·4335 H.

H	P	p	H	P	p	H	P	p	H	P	p
1	0 56	0 43	7	3 90	3 03	18	10 03	7·80	50	27·87	21 67
2	1 11	0·87	8	4 46	3 47	20	11·15	8 67	60	33 44	26·01
3	1 67	1·30	9	5 02	3·90	25	13 93	10·84	70	39·02	30·35
4	2 23	1 73	10	5·57	4 34	30	16·72	13 01	80	44·59	34·68
5	2 79	2 17	12	6 69	5·20	35	19 51	15·18	90	50 17	39 02
6	3 34	2 60	15	8 36	6 50	40	22 30	17 34	100	55·74	43·35

III.—Velocity of Water.

V = Theoretical velocity in feet per second.
g = Force of gravity = 32·2 feet per second.

$2g = 64{\cdot}4 \qquad \sqrt{2g} = 8\ 025 \qquad \frac{1}{2g} = 0\ 0155$

H = Head of water in feet.

$V = \sqrt{2g\,H} = 8{\cdot}025\,\sqrt{H}.$

$H = \frac{V^2}{2g} = 0\ 0155\ V^2.$

Theoretical Velocity due to Different Heads in Feet per Second.

Feet per Second

Head in Feet	0	1	2	3	4	5	6	7	8	9
0	—	8 025	11 35	13·90	16·05	17·94	19 66	21·23	22·69	24
10	25·38	26 62	27 80	28 93	30·00	31 08	32·10	33 09	34·05	34
20	35·89	36·77	37·64	38·48	39·31	40·12	40·92	41·70	42·40	43
30	43 95	44·68	45 40	46 10	46·79	47 47	48 15	48·82	49·47	50
40	50 75	51·39	52 01	52·62	53 23	53·83	54 43	55·02	55·60	56
50	56·74	57 31	57·87	58 42	58·97	59·51	60·05	60·59	61 11	61
60	62 16	62·68	63 19	63·69	64·20	64·70	65·19	65·69	66 18	66
70	67·14	67·62	68 09	68 57	69·03	69·50	69·96	70·42	70·87	71
80	71·78	72·23	72·67	73 11	73 55	74 00	74 42	74 85	75 28	75
90	76·13	76 55	76·97	77·39	77 81	78 22	78 63	79 04	79 44	79
100	80 25	—	—	—	—	—	—	—	—	–

IV. Discharge of Water from Sluices.

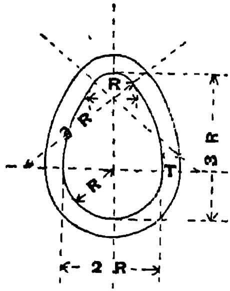
V = Theoretical velocity in feet per second (*vide* Table III) due to the head of water (from surface of water upstream to that downstream);
A = Area of aperture in square feet;
E = Velocity of efflux in feet per second;
K = Coefficient for different orifices;
Q = Discharge in cubic feet per second;

$E = V\,K. \qquad Q = E\,A = V\,K\,A.$

For large sluices with pointed piers . K = 0·95.
For small sluices . . . . . K = 0·85.
For pipe sluices, bore = $\frac{1}{4}$ to $\frac{1}{24}$ length. K = 0·77 to 0·73.

V. Areas, &c., of Outlet Culverts.[1]

(Proportion of Height to Extreme Width Inside 3 to 2.)



Size of Culvert in Inches.	Internal Area of Culvert in Square Feet.	Hydraulic Mean Depth, flowing full, in Feet.	Size of Culvert in Inches	Internal Area of Culvert in Square Feet	Hydraulic Mean Depth, flowing full, in Feet
36 × 54	10 337	0·869	56 × 84	25·012	1 352
38 × 57	11·517	0·917	58 × 87	36·830	1·400
40 × 60	12·761	0 966	60 × 90	28·713	1·449
42 × 63	14·069	1·014	62 × 93	30·665	1·497
44 × 66	15·410	1 062	64 × 96	32·668	1·545
46 × 69	16 877	1 111	66 × 99	34·742	1 593
48 × 72	18 376	1·159	68 × 102	36·880	1·642
50 × 75	19·939	1 207	70 × 105	39 081	1·690
52 × 78	21 566	1·255	72 × 108	41·346	1.738
54 × 81	23·257	1 304			

Thickness, T. of Masonry Culvert Rings.

	Culverts under 48″ × 72″	Culverts 48″ × 72″ to 60″ × 90″	Culverts 60″ × 90″ and over.
For culverts under dams :—			
Less than 50 feet high	1′ 3″	1′ 9″	2′ 0″
More than 50 feet high	1′ 6″	2′ 0″	2′ 3″

[1] "Sanitary Engineering," by Baldwin Latham, 2nd edition, Table No. 30, p, 135.

APPENDIX 26.

STATISTICAL INFORMATION REQUIRED FOR A PROJECT.

(*Vide* Chapter V, paragraph 242 (*a*), page 311.)

Consecutive No.	Items	Unit	Amount
	A.—ENGINEERING.		
	I. General.		
1	Area of catchment (enter actual or equivalent)	sqr. miles	
2	Average annual monsoon rainfall . .	inches	
3	Estimated percentage of run-off to average monsoon rainfall	per cent.	
4	Estimated depth of run-off from average monsoon rainfall (2) × (3) . . .	inches	
5	Estimated average annual yield from catchment (1) × (4) ÷ 12 . . .	mill. cub. ft.	
6	Percentage of average annual yield to gross available full-supply storage capacity of reservoir (5) and (*II*. 5) . . .	per cent.	
7	Percentage of number of years of ordinary minimum monsoon rainfall . . .	do.	
8	Ordinary minimum annual monsoon rainfall	inches	
9	Estimated percentage of run-off of (8) .	per cent.	
10	Estimated depth of run-off from ordinary minimum monsoon rainfall (8) × (9) .	inches	
11	Estimated yield from ordinary minimum monsoon rainfall (1) × (10) ÷ 12 . .	mill. cub. ft.	
12	Percentage of ordinary minimum yield to gross available full-supply storage capacity of reservoir (11) and (*II*. 5) .	per cent.	
13	Depth of run-off required to produce gross available full-supply storage of reservoir (*II*. 5) ÷ (1) × 12	inches	
14	Estimated average yield of feed channel during monsoon	mill. cub. ft.	
15	Estimated average yield of feed channel during fair weather	do.	
16	Estimated amount of fair-weather flow of impounded stream	do.	
17	Fall of river bed in reservoir full-supply limits	ft. per mile	

TABLE—*continued*

Consecutive No	Items	Unit	Amount
	A.—ENGINEERING—*continued.*		
	II. Reservoir.		
1	Area of reservoir at full-supply level	mill. sq. ft. / acres	
2	Area of reservoir at outlet sill level .	do. do.	
3	Mean area of reservoir ½ [(1) + (2)] . .	do do	
4	Total full-supply storage capacity .	mill. cub. ft.	
5	Gross available full-supply storage capacity above outlet sill.	do. do.	
6	Loss of storage by evaporation and absorption (3) × (9) during year . . .	do. do.	
7	Net full-supply storage available for irrigation (5) — (6)	do. do.	
8	Average depth of gross available full-supply storage (5) ÷ (3)	feet	
9	Estimated depth lost by evaporation and absorption on mean area of reservoir (3)	do.	
10	Estimated average draw-off from reservoir during monsoon	mill. cub ft.	
11	Estimated average balance in reservoir at end of monsoon	do. do.	
12	Total supply available for irrigation (*vide VI.* 12) = (*I.* 15 + 16) + (10) + (11) — (6)	do. do.	
	III. Dam.		
1	Length of dam at top	feet	
2	Maximum height of dam above ground level	do.	
3	Top of dam	R.L.	
4	High-flood level . . .	do.	
5	Full-supply level { temporary . . .	do.	
	permanent . . .	do.	
6	Outlet sill level	do.	
7	Lowest ground level of dam	do.	
8	Top-width of dam	feet	
9	Upstream slope of dam	ratio	
10	Downstream slope of dam	do.	
11	Maximum depth of puddle treach . . .	feet	
12	Bottom-width of puddle trench . . .	do.	
13	Side-slopes of puddle trench	ratio	
14	Top of pitching	R.L.	

TABLE—*continued*

Consecutive No	Items	Unit	Amount.
	A —ENGINEERING—*continued.*		
	III. Dam—continued.		
15	Bottom of pitching	R.L.	
16	Maximum thickness of pitching . .	feet	
17	Maximum depth of full-supply (5) — (7) .	do.	
18	Depth from full-supply to outlet still (5) — (6)	do.	
	IV. Waste-Weir.		
1	Description of waste-weir (clear overfall, drowned or channel)	—	
2	Total length of waste-weir . . .	feet	
3	Net length of overfall of weir crest . .	do.	
3B	Arcade arches	No. and span	
4	Under-sluices	No. and size	
5	Automatic gates	do. do.	
5B	Depth stored by temporary crest . .	feet	
6	Calculated high-flood depth over waste-weir wall crest	feet	
7	Calculated high-flood discharge of waste-weir wall crest	cusecs.	
8	Do. do. do. of under-sluices .	do.	
9	Do. do. do. of automatic gates .	do.	
10	Do. do. do. of whole weir (7) + (8) + (9)	do.	
11	Estimated high-flood discharge from catchment	do.	
12	Calculated rate of run-off from catchment of waste-weir high-flood discharge (10) .	in. per hour	
13	Estimated rate of maximum run-off from catchment (11)	do. do.	
	V. Outlet.		
1	Description of outlet (culvert under dam, headwall, &c.)	—	
2	Sluices	No. and size	
3	Discharging capacity of sluices at high-flood level	cusecs.	
4	Do. do. do. at full supply level	do.	
5	Head required to give canal full-supply discharge (*VI.* 6)	feet	

TABLE—*continued.*

Consecutive No	Items	Unit.	Amount
	A.—ENGINEERING—*continued*		
	VI. Canal and Irrigation.		
1	Description of headworks (reservoir outlet, pick-up weir, &c)	—	
2	Distance of headworks from reservoir .	miles	
3	Estimated loss in transit . . .	per cent	
4	Length of main canal	miles	
5	Length of distributaries	do.	
6	Initial full-supply discharge of canal .	cusecs.	
7	Do. full-supply depth do. . .	feet	
8	Do. bed-level do . .	R.L	
9	Do. bed-width do. . .	do.	
10	Estimated average area of irrigated crops— Perennial. .... Rabi ..... Hot weather .... Monsoon.	Total Acres	
11	Estimated duty of water at canal head— Perennial. . .. . Rabi. . . Hot weather ... .. Monsoon	acres per cusec.	
12	Estimated supply required for irrigated crops [*vide II.* (12) and (10) and (11)]— Perennial. .... .. Rabi .... Hot weather. . ... Monsoon.	Total mill cub. ft	
13	Estimated irrigation assessment rates per acre— Perennial. ... ... Rabi . . Hot weather .. . . . Monsoon.	Rs.	
14	Estimated gross irrigation revenue (10) × (13)— Perennial .. Rabi. .... Hot weather .... Monsoon	Total Rs.	
15	Average acreage irrigated per mile of canal (10) ÷ (4)	acres	

Consecutive No	*VII. Land.*	Reservoir.	Canal.	Total
1	Total area to be acquired in acres			
2	Culturable area do do.			
3	Percentage of (2) to (4)		per cent.	
4	Area irrigable by project in average years		acres	
5	Do. do. do. in years of minimum rainfall		do.	

TABLE—*continued.*

Consecutive No	Items	Unit	Amount
	A.—ENGINEERING—*continued.*		
	VII. Land—continued.		
6	Culturable area commanded by project .	acres	
7	Percentage of (4) to (6)	per cent.	
8	Percentage of (5) to (6)	do.	
	B.—FINANCIAL.		
1	Estimated cost of dam	Rs.	
2	Estimated cost of waste-weir(s) . .	,,	
3	Estimated cost of outlet(s) . . .	,,	
4	Estimated cost of land compensation .	,,	
5	Estimated cost of buildings and miscellaneous expenditure	,,	
6	Total estimated cost of reservoir (1) to (5)	,,	
7	Estimated cost of feed channel . .	,,	
8	Estimated cost of canal	,,	
9	Total estimated cost of project for works (6) to (8)	,,	
10	Percentage cost of dam (1) to cost of reservoir (6)	per cent.	
11	Percentage cost of waste-weir (2) to cost of reservoir (6)	,,	
12	Percentage cost of outlet (3) to cost of reservoir (6)	,,	
13	Percentage cost of land compensation (4) to cost of reservoir (6)	,,	
14	Percentage cost of reservoir (6) and feed channel (7) to cost of project (9) . .	,,	
15	Percentage cost of canal (8) to cost of project (9)	,,	
16	Establishment at 23 per cent. on (9) less cost of excluded items (Rs.) . .	Rs.	
17	Tools and plant at $1\frac{1}{2}$ per cent. on (9) less do. do.	,,	
18	Total direct charges (9) + (16) + (17) .	,,	
19	Capitalization of abatement of land revenue	,,	
20	Leave and pension allowances at 14 per cent. on (16)	,,	

TABLE—*continued.*

Consecutive No.	Items	Unit	Amount
	B.—FINANCIAL—*continued.*		
20B	Interest on (9) at 2 per cent on year's expenditure and 4 per cent. on previous expenditure	Rs.	
21	Total indirect charges (19) + (20) + 20B .	,,	
22	Grand Total, direct and indirect charges (18) + (21)	,,	
23	Area irrigable (*vide VI.* 10)	acres	
24	Gross Revenue (*vide VI.* 14) *plus* miscellaneous receipts	Rs.	
25	Deduct working expenses at Rs. per acre (23)	,,	
26	Net revenue (24) — (25) . . .	,,	
27	Percentage return of (26) on (22) . .	per cent.	
28	Estimated rate of gross available full-supply storage (6) + (7) ÷ (*II.* 5) .	Rs per mill. cub ft.	
29	Estimated rate of direct charges per acre irrigable by project (18) ÷ (23) . .	Rs.	
30	Estimated rate of direct and indirect charges per acre irrigable by project (22) ÷ (23)	,,	
31	Normal value of work available for famine relief	,,	
32	Relief labourers for whom employment can be found on works for 6 months .	No.	

INDEX.

NOTE.—The figures refer to pages

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PLATE No. I.

DIAGRAM
SHOWING APPROXIMATE AMOUNTS OF PERCENTAGE OF RUN-OFF TO RAINFALL, DEPTH OF RUN-OFF DUE TO RAINFALL & YIELD OF RUN-OFF FROM GOOD, AVERAGE & BAD CATCHMENTS.

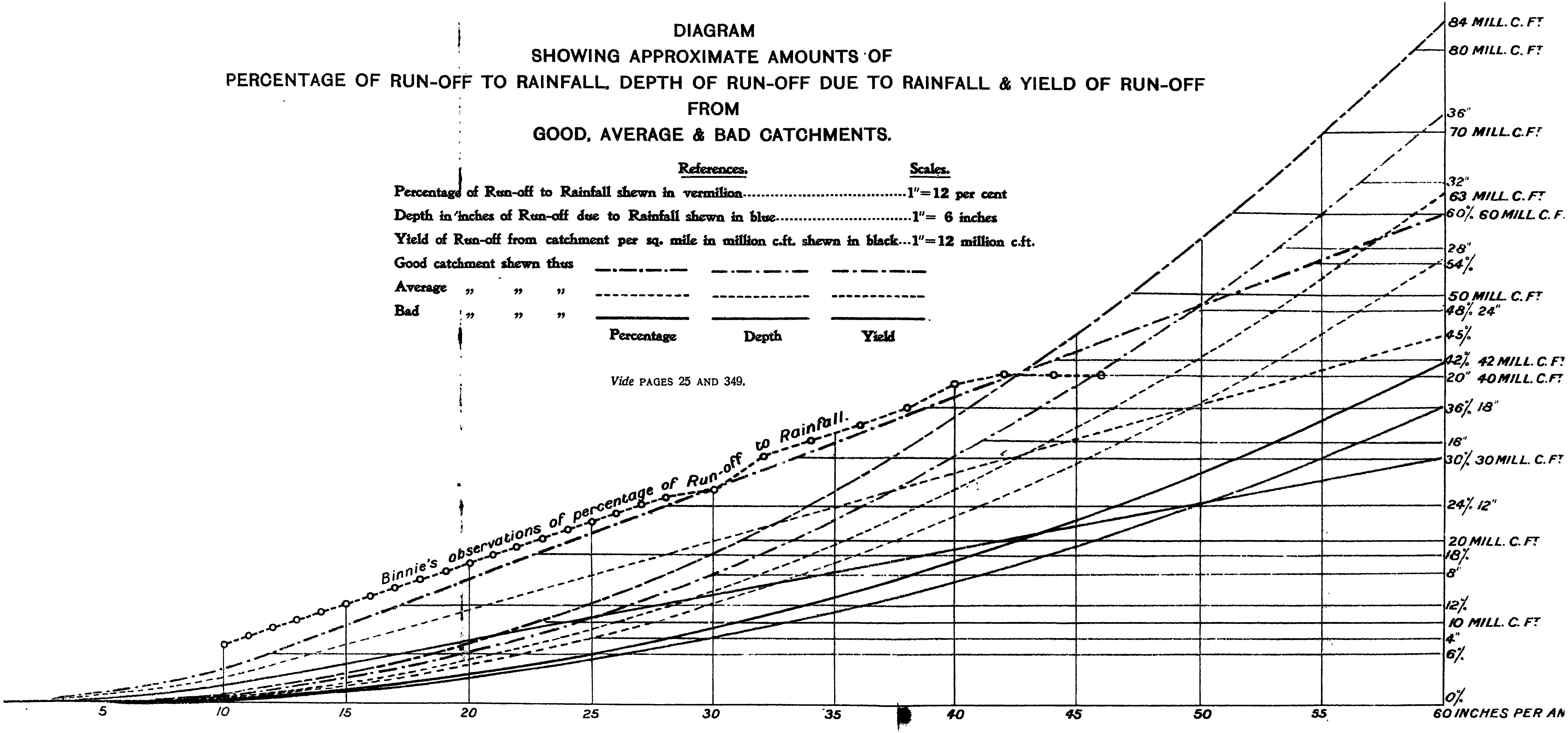


PLATE No. 2.

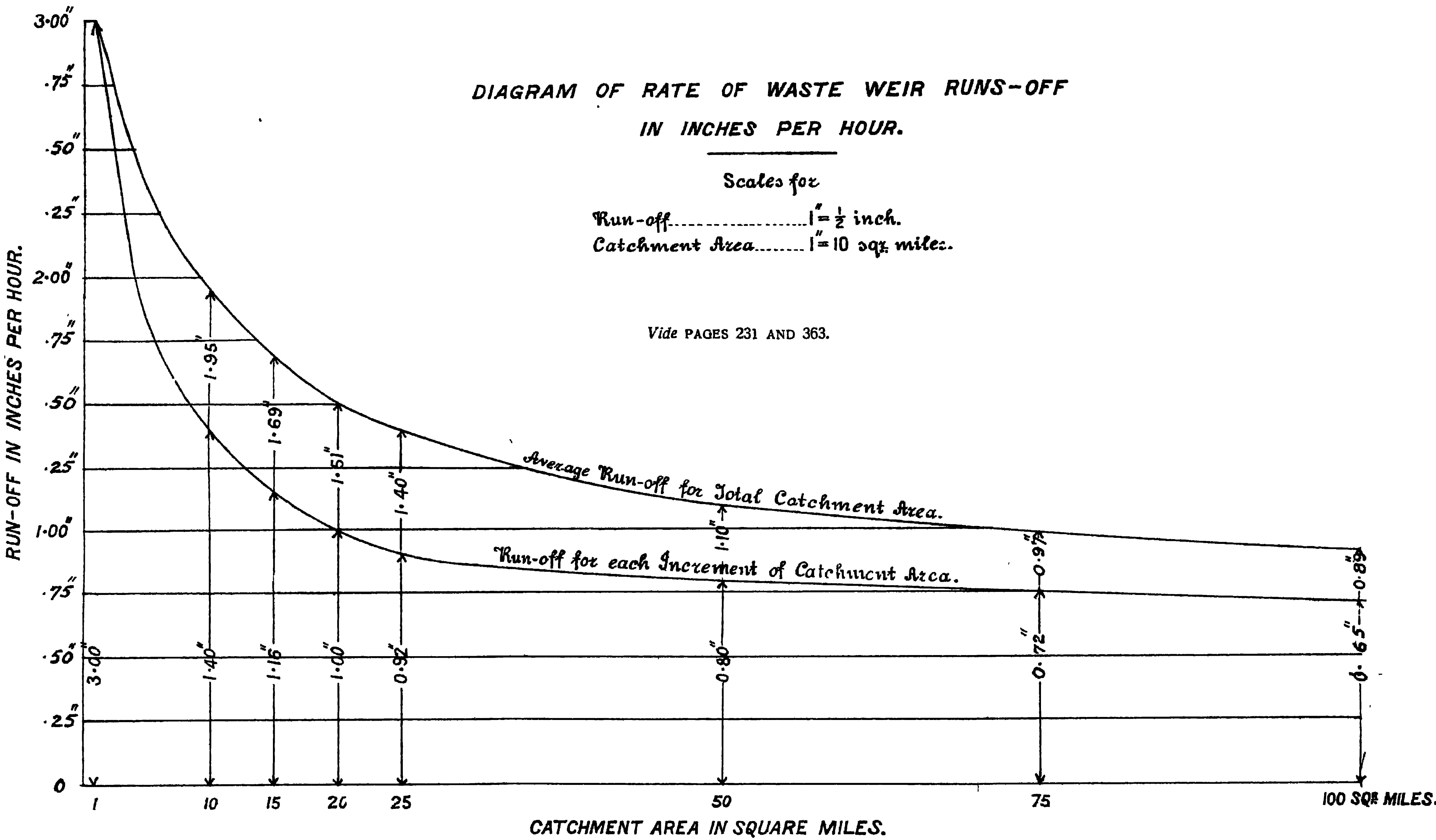


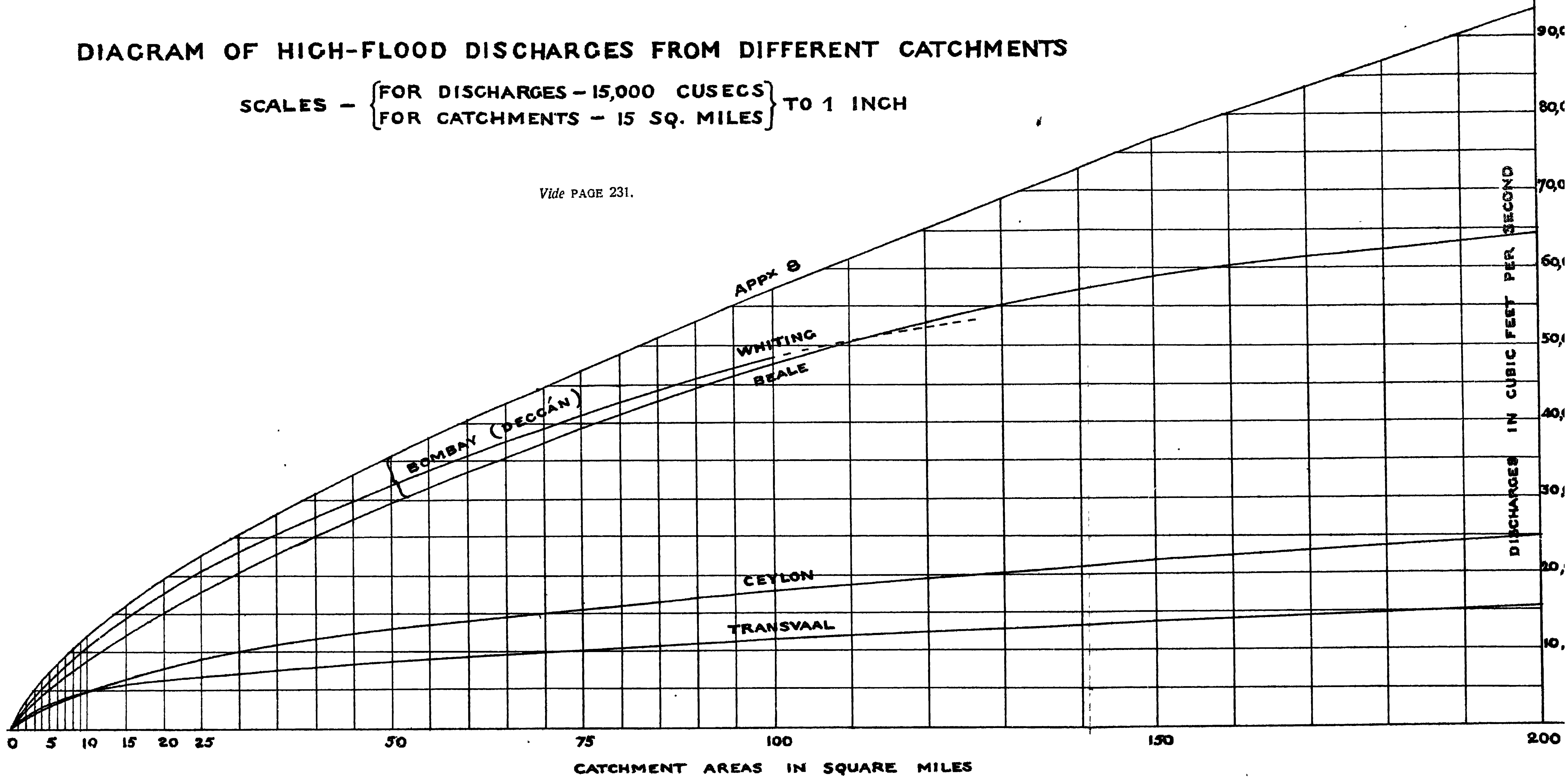
DIAGRAM OF RATE OF WASTE WEIR RUNS-OFF
IN INCHES PER HOUR.
Scales for
Run-off.......................1" = ½ inch.
Catchment Area........1" = 10 sqr. miles.
Vide PAGES 231 AND 363.
Average Run-off for Total Catchment Area.
Run-off for each Increment of Catchment Area.
RUN-OFF IN INCHES PER HOUR.
3·00"
·75"
·50"
·25"
2·00"
·75"
·50"
·25"
1·00"
·75"
·50"
·25"
0
1·95"
1·69"
1·51"
1·40"
1·10"
0·97"
0·89"
3·00"
1·40"
1·16"
1·00"
0·92"
0·80"
0·72"
0·65"
1
10
15
20
25
50
75
100 SQR. MILES.
CATCHMENT AREA IN SQUARE MILES.

PLATE N° 2^A^

DIAGRAM OF HIGH-FLOOD DISCHARGES FROM DIFFERENT CATCHMENTS

SCALES — { FOR DISCHARGES — 15,000 CUSECS / FOR CATCHMENTS — 15 SQ. MILES } TO 1 INCH

Vide PAGE 231.



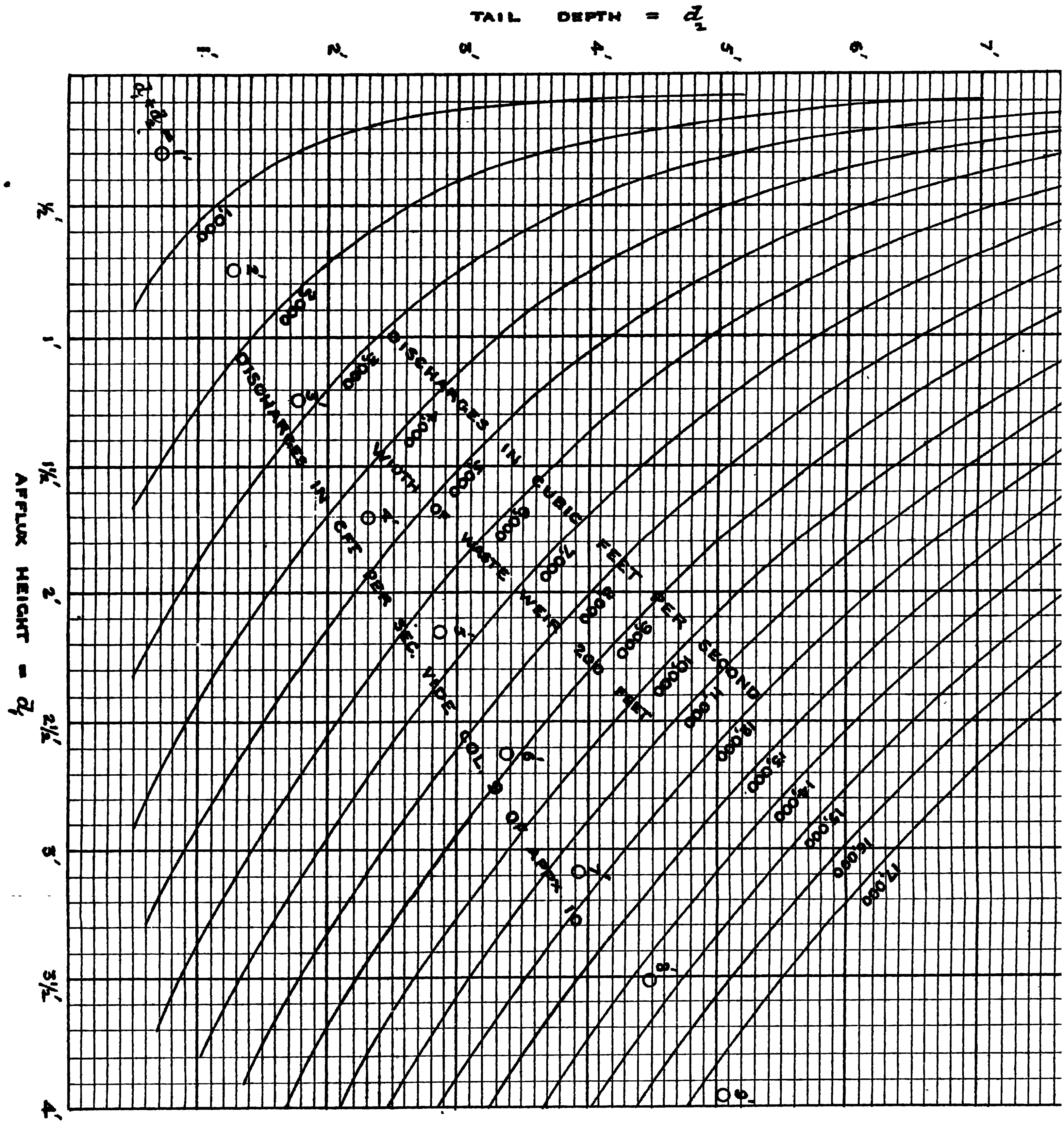


PLATE N° 2[B]

DIAGRAM OF WASTE WEIR DISCHARGES WITH VARYING TAIL CHANNEL DEPTHS AND AFFLUX HEIGHT

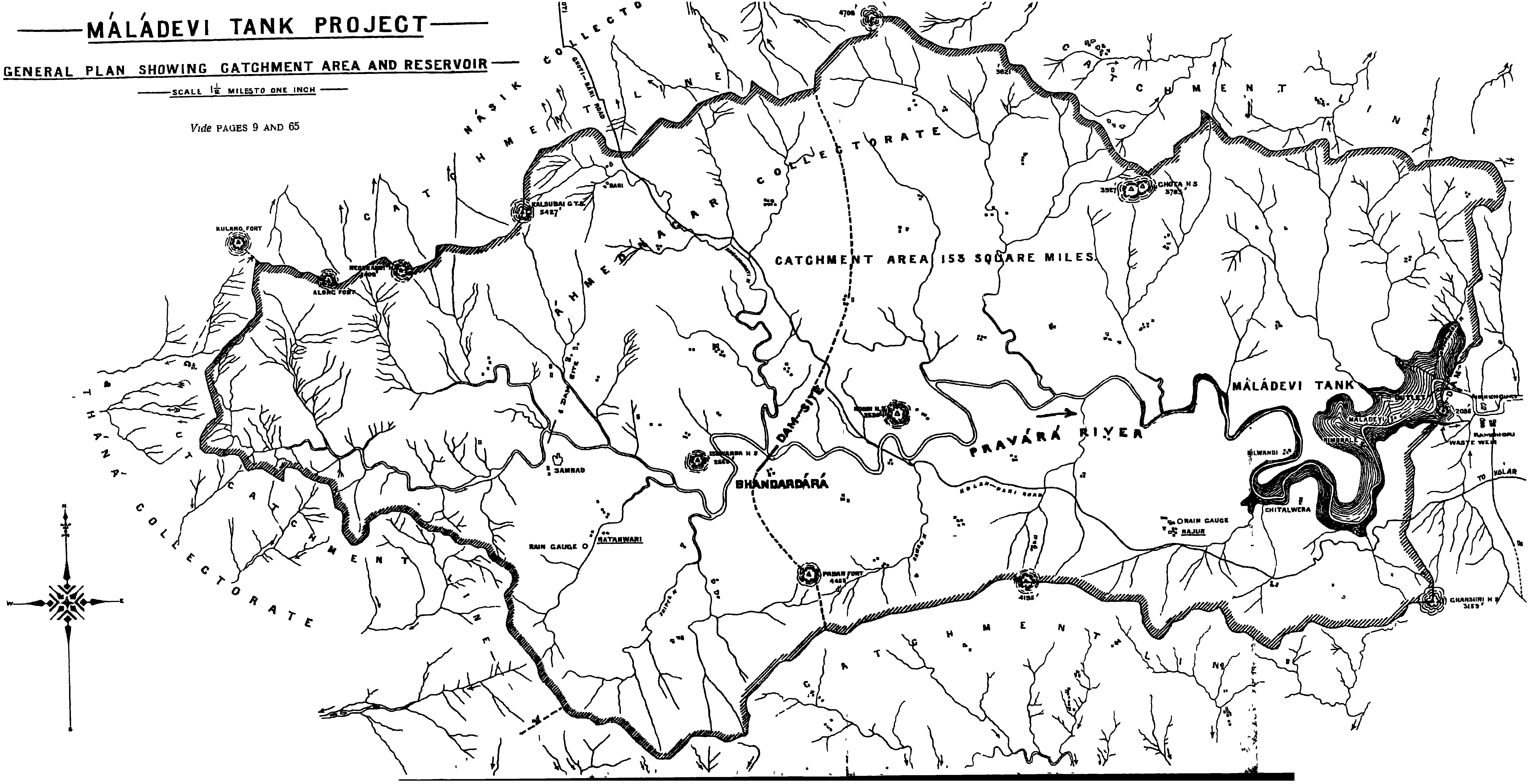
Vide PAGE 240.

MÁLADEVI TANK PROJECT

GENERAL PLAN SHOWING CATCHMENT AREA AND RESERVOIR

SCALE 1½ MILES TO ONE INCH

Vide PAGES 9 AND 65



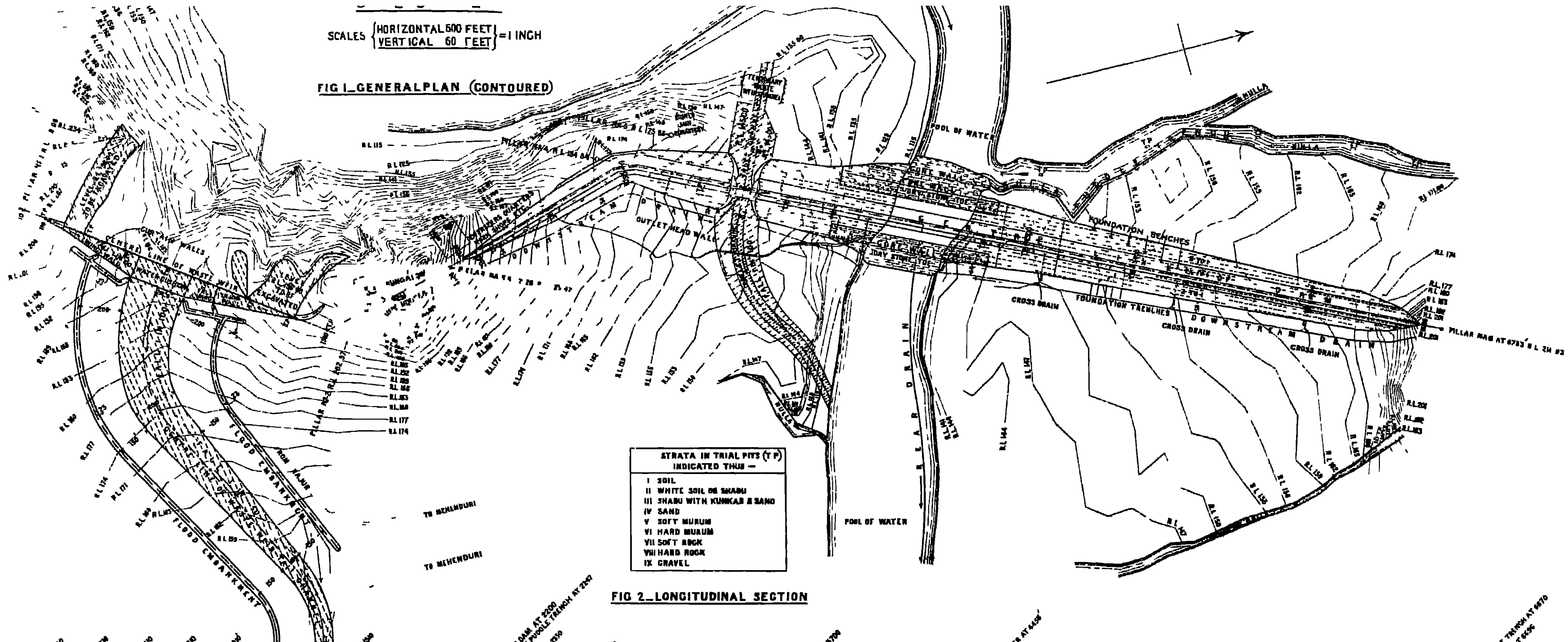
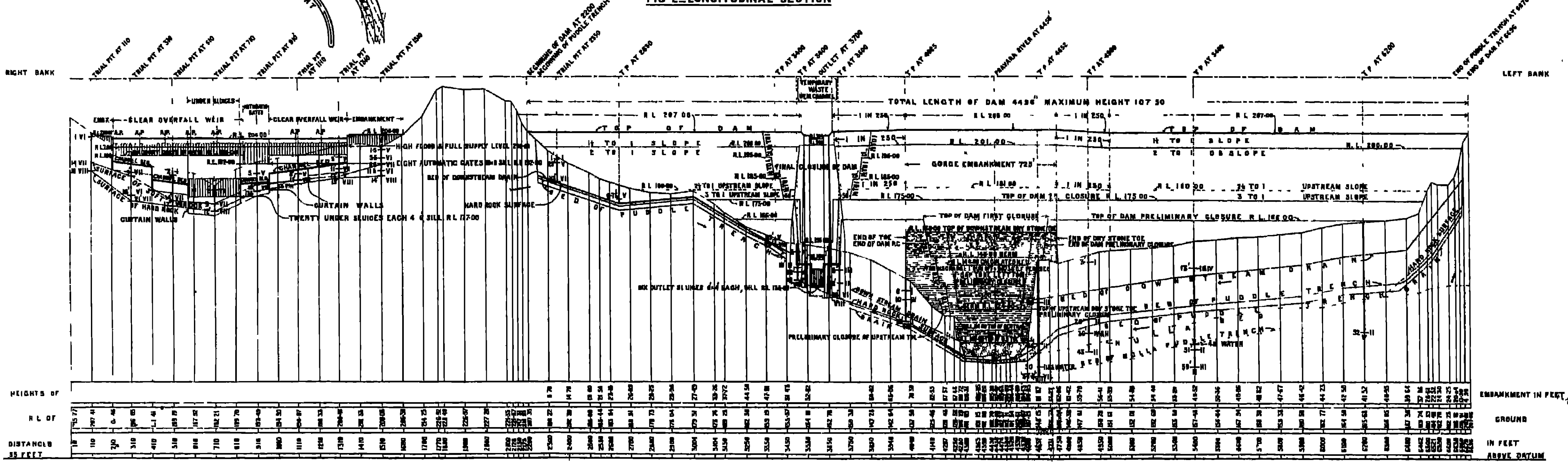


FIG 2_LONGITUDINAL SECTION



Vide PAGES 66, 141, 148, 174, 186, 192, 244, 249 AND 265

—MÁLÁDEVI TANK PROJECT—

PLATE Nº 5

—CROSS SECTIONS OF DAM EMBANKMENT—

—SCALE 60 FEET = 1 INCH—

FIG_1

—ENLARGED VIEW OF TOP OF DAM—

—SCALE 10 FT. TO 1 INCH—

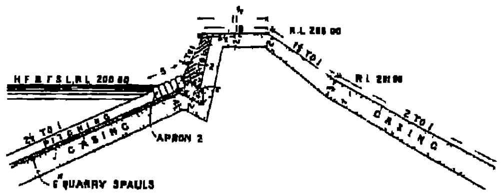


FIG_2

—TYPE SECTION FOR GORGE EMBANKMENT—

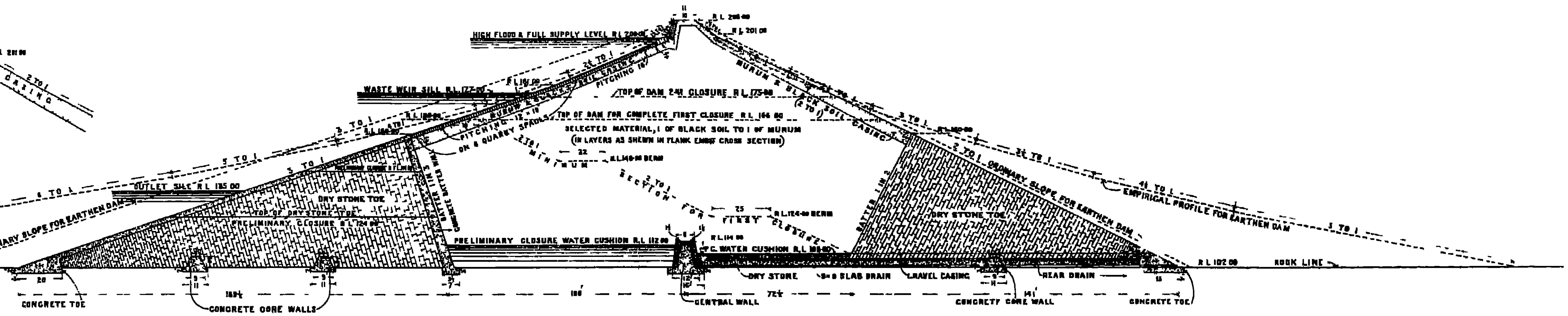


FIG_3

—SECTION OF NULLA PUDDLE TRENCH—

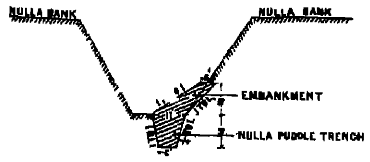


FIG_4

—TYPE SECTION FOR FLANK EMBANKMENT—

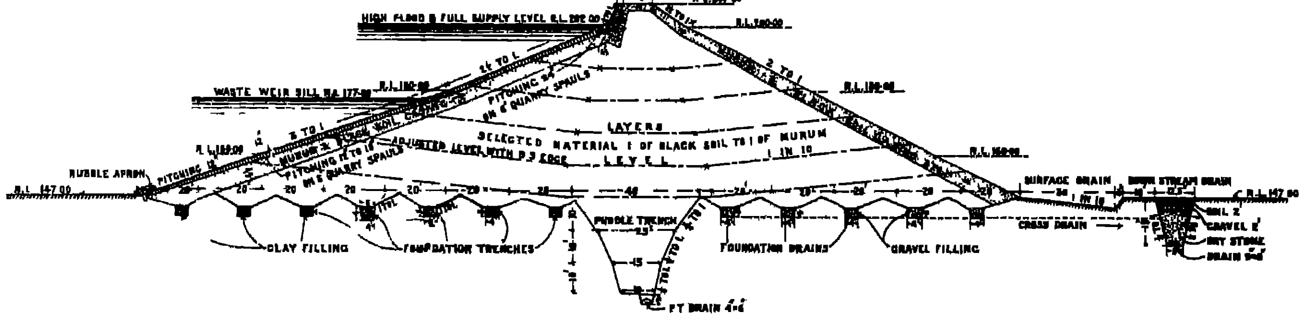


FIG_5

—CENTRAL WALL ELEVATION—

—SCALE 60 FT. TO 1 INCH—

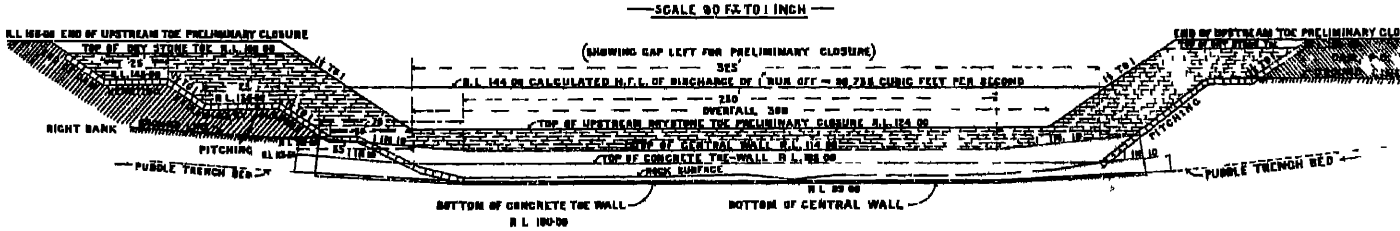


FIG_6

—PLAN—

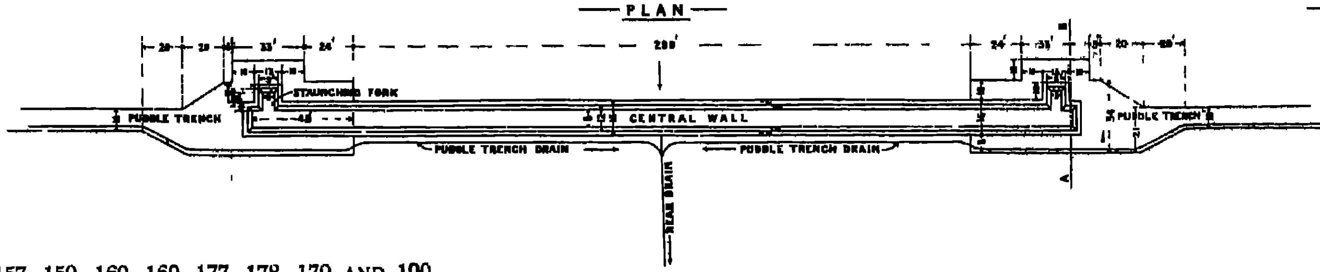


FIG_7

—CROSS SECTION AT

Vide PAGES 91, 97, 102, 134, 141, 148, 157, 159, 160, 169, 177, 178, 179 AND 190

V SOFT MURUM
VI HARD MURUM
VII SOFT ROCK
VIII HARD ROCK
IX GRAVEL

SCALES {HORIZONTAL 300 FEET / VERTICAL 30 FEET} = 1 INCH

FIG_1 LONGITUDINAL SECTION

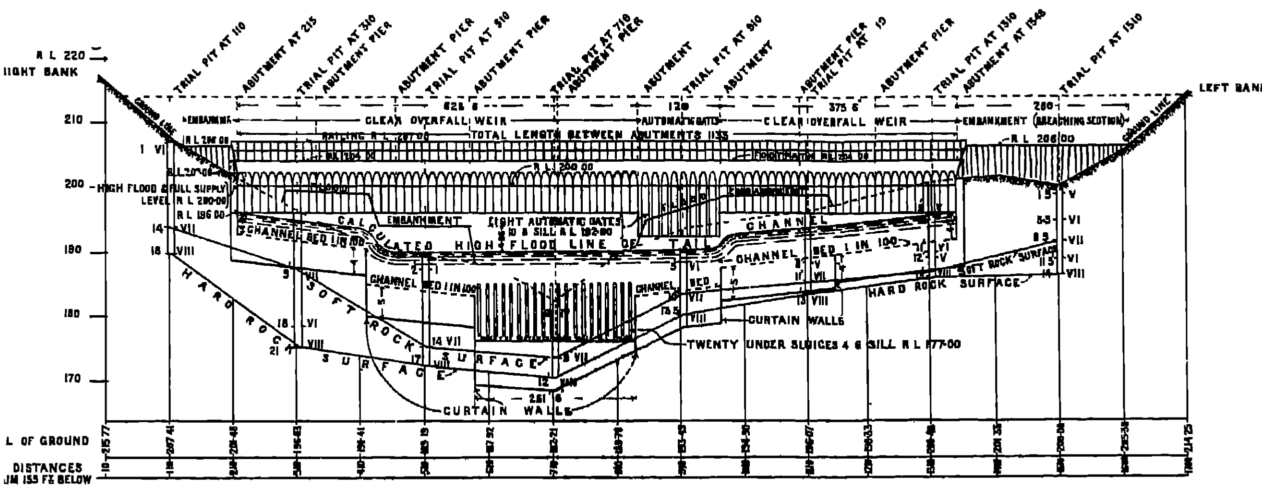
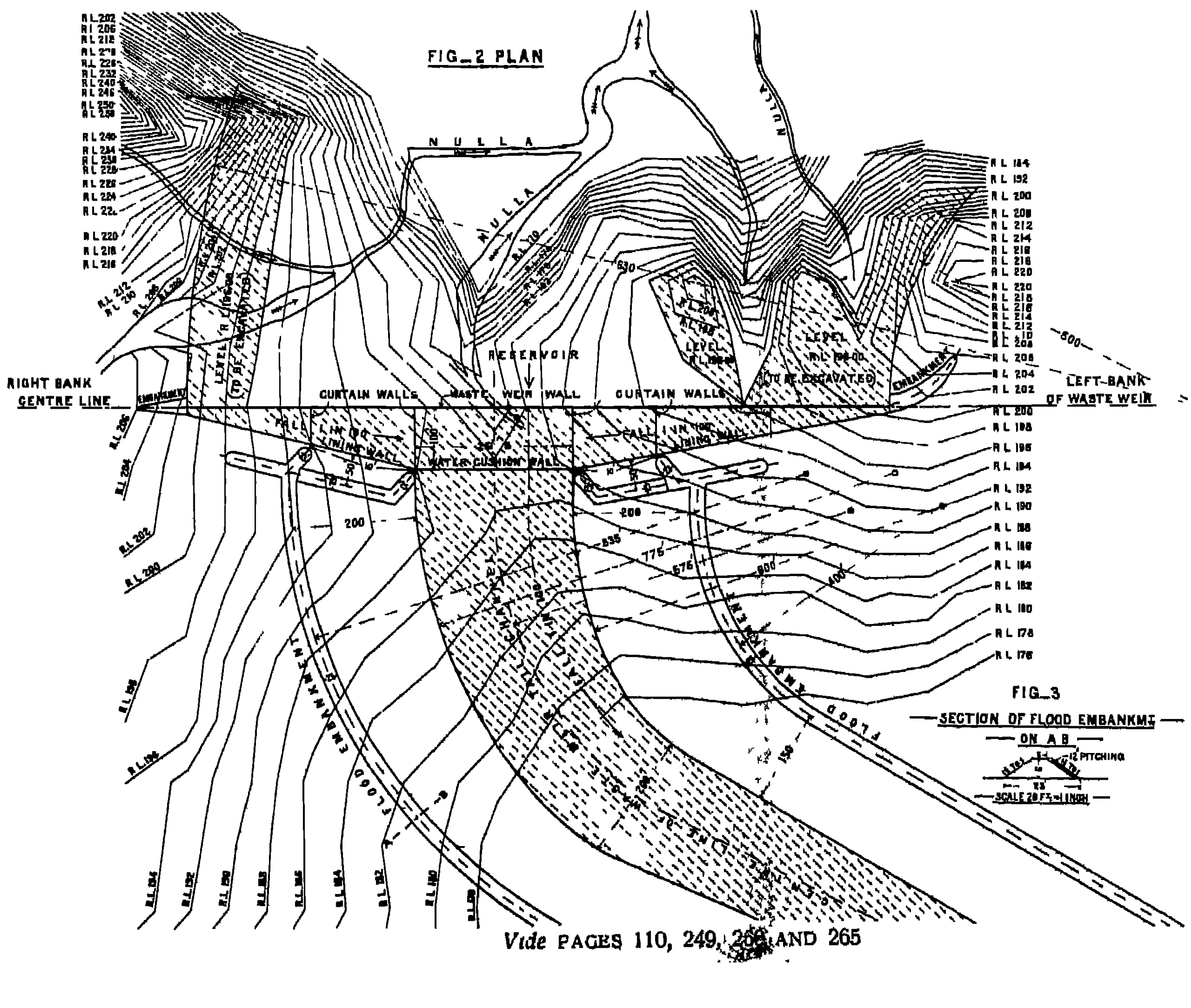


FIG_2 PLAN

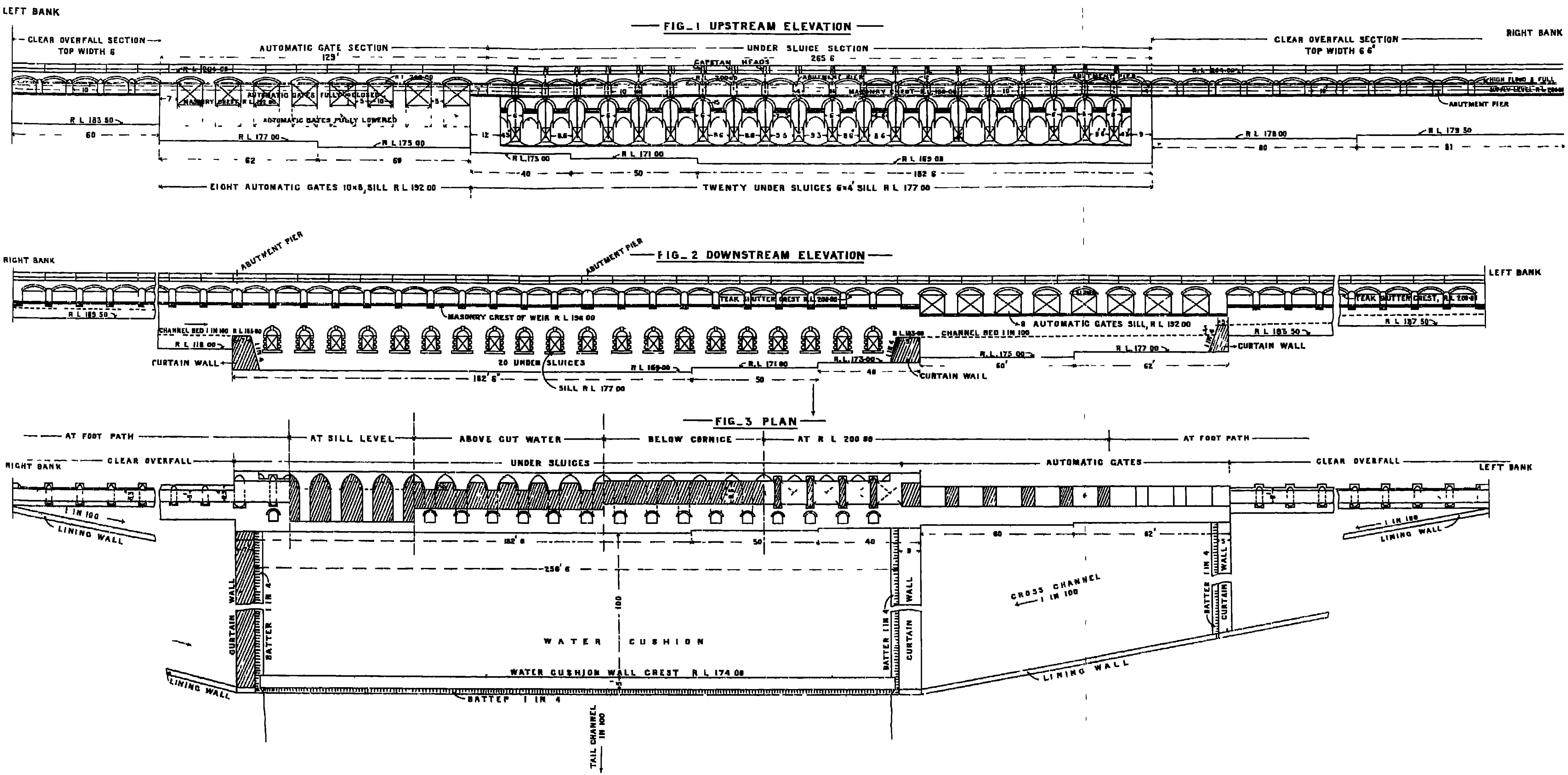


Vide PAGES 110, 249, 259 AND 265

—WASTE WEIR—

—ENLARGED GENERAL DRAWINGS—

—SCALE 50 FT. TO ONE INCH—



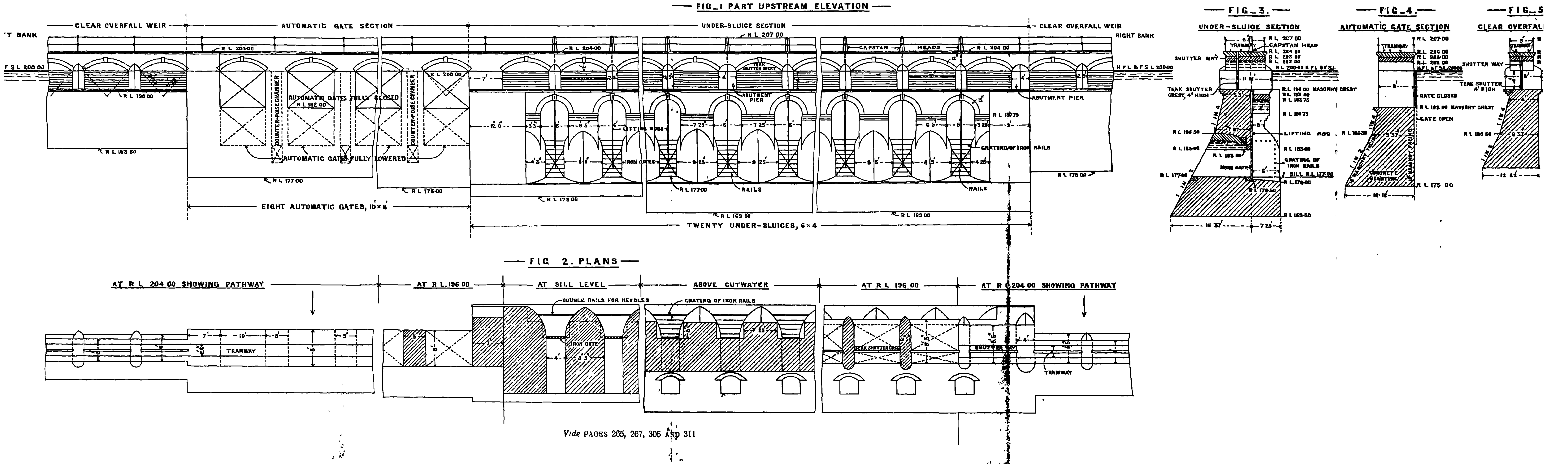
Vide PAGES 191, 265 AND 267.

MALADEVI TANK PROJECT

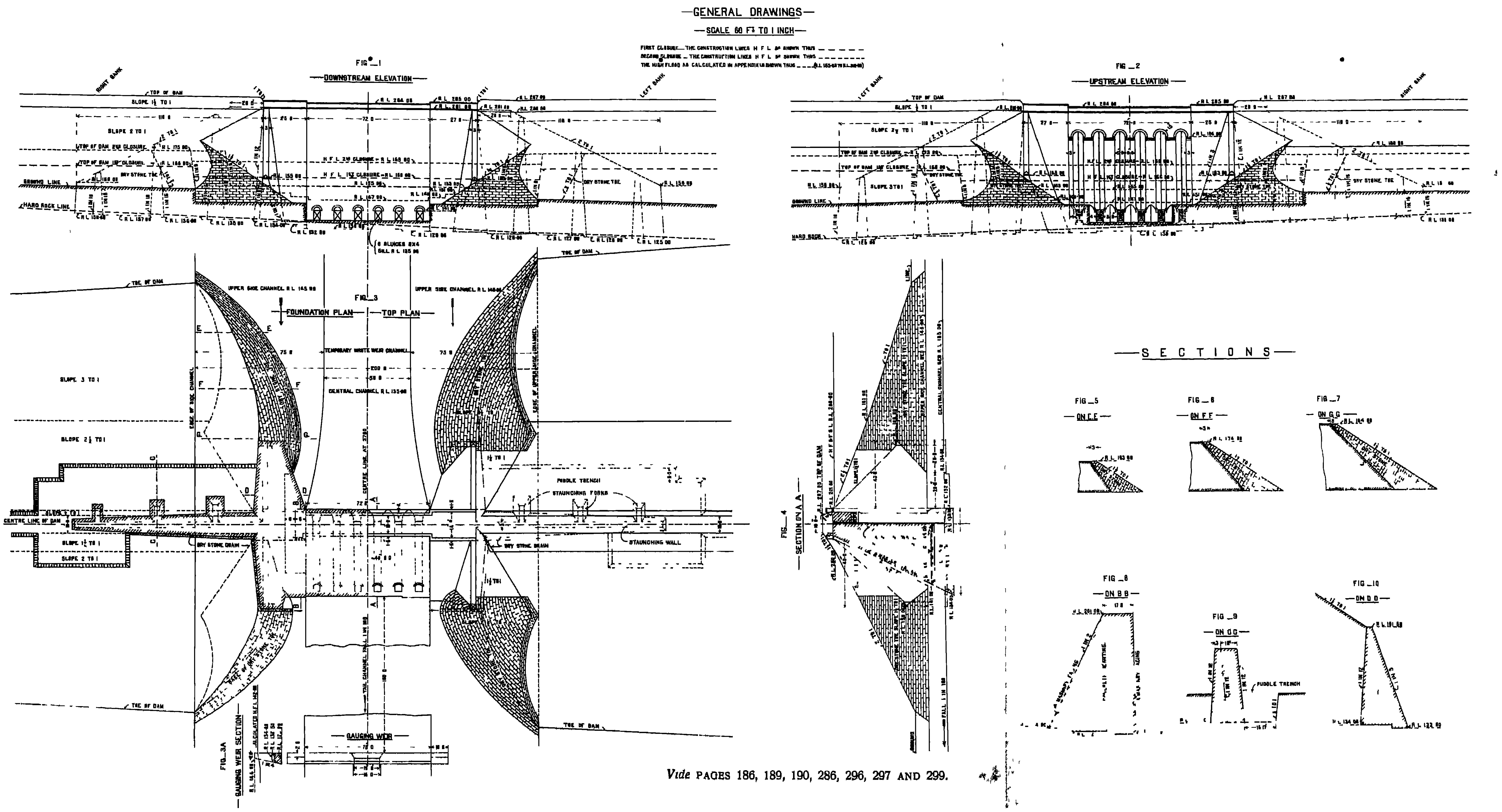
WASTE WEIR

DETAILED DRAWINGS

SCALE 15 FT TO ONE INCH



Vide PAGES 265, 267, 305 AND 311



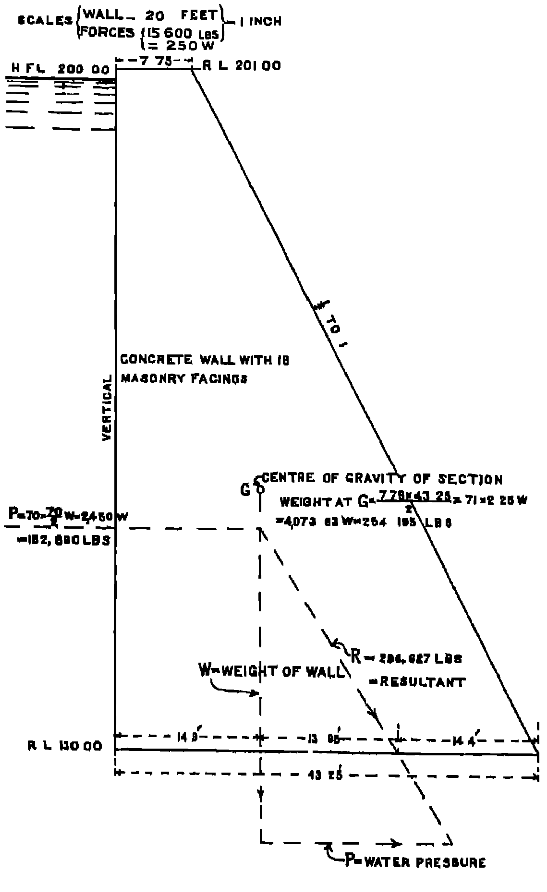
Vide PAGES 186, 189, 190, 286, 296, 297 AND 299.

DETAILED DRAWINGS

SCALE 25 FT TO ONE INCH

FIG_1 STABILITY DIAGRAM

MEAN SECTION



W= WEIGHT OF WATER PER C FT = 62 4 LBS
WEIGHT OF WALL PER C FT = 2 25 W=140 4 LBS

FIG_2 PART UPSTREAM ELEVATION

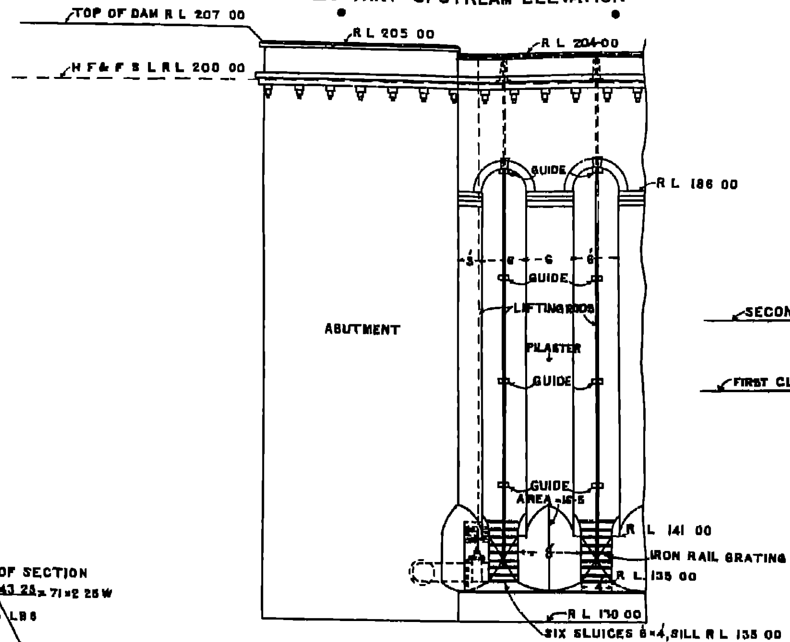


FIG_3 SECTION ON A B

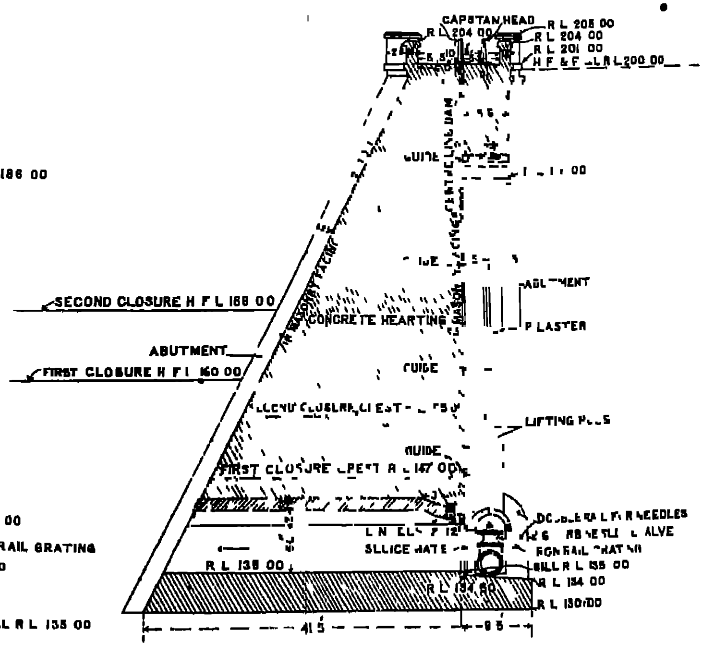
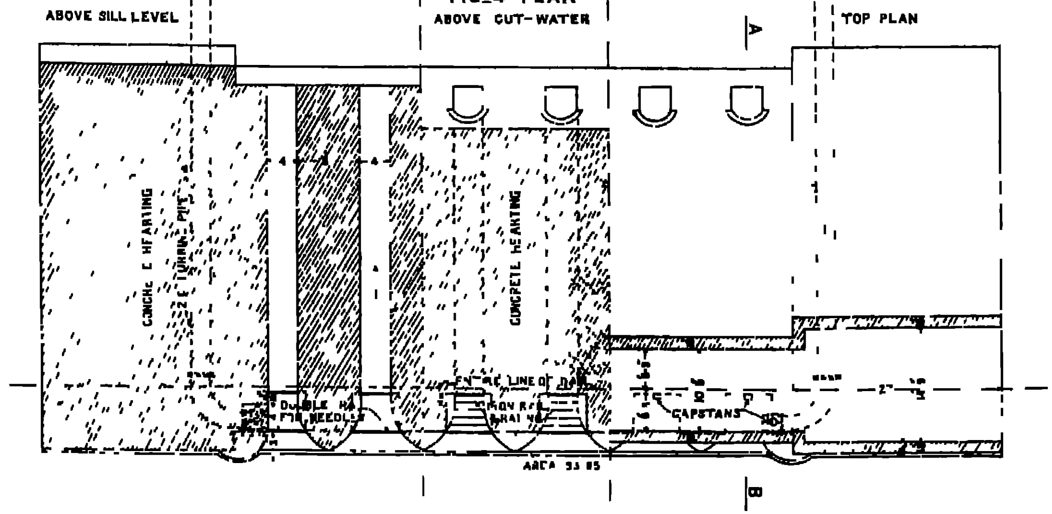
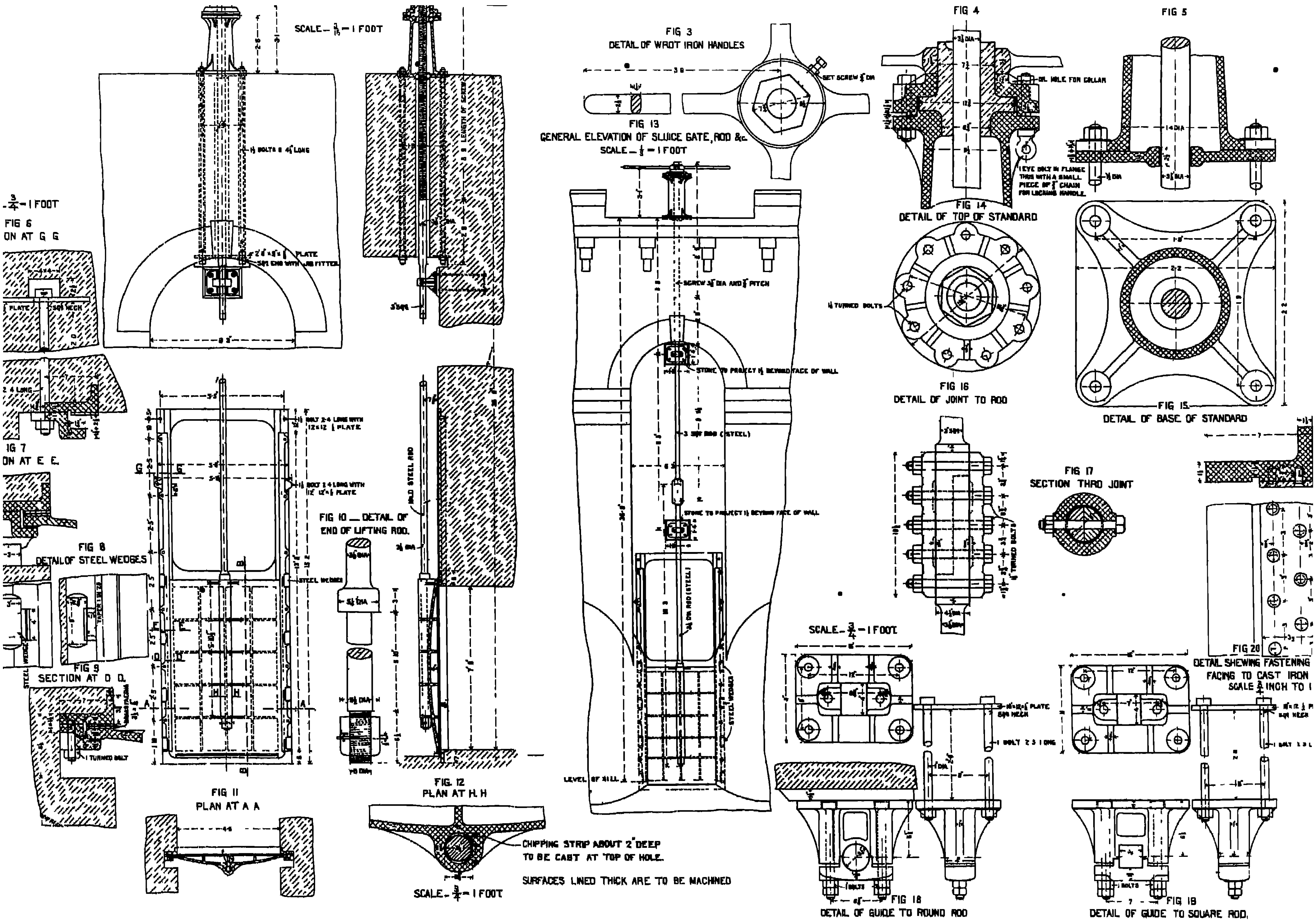


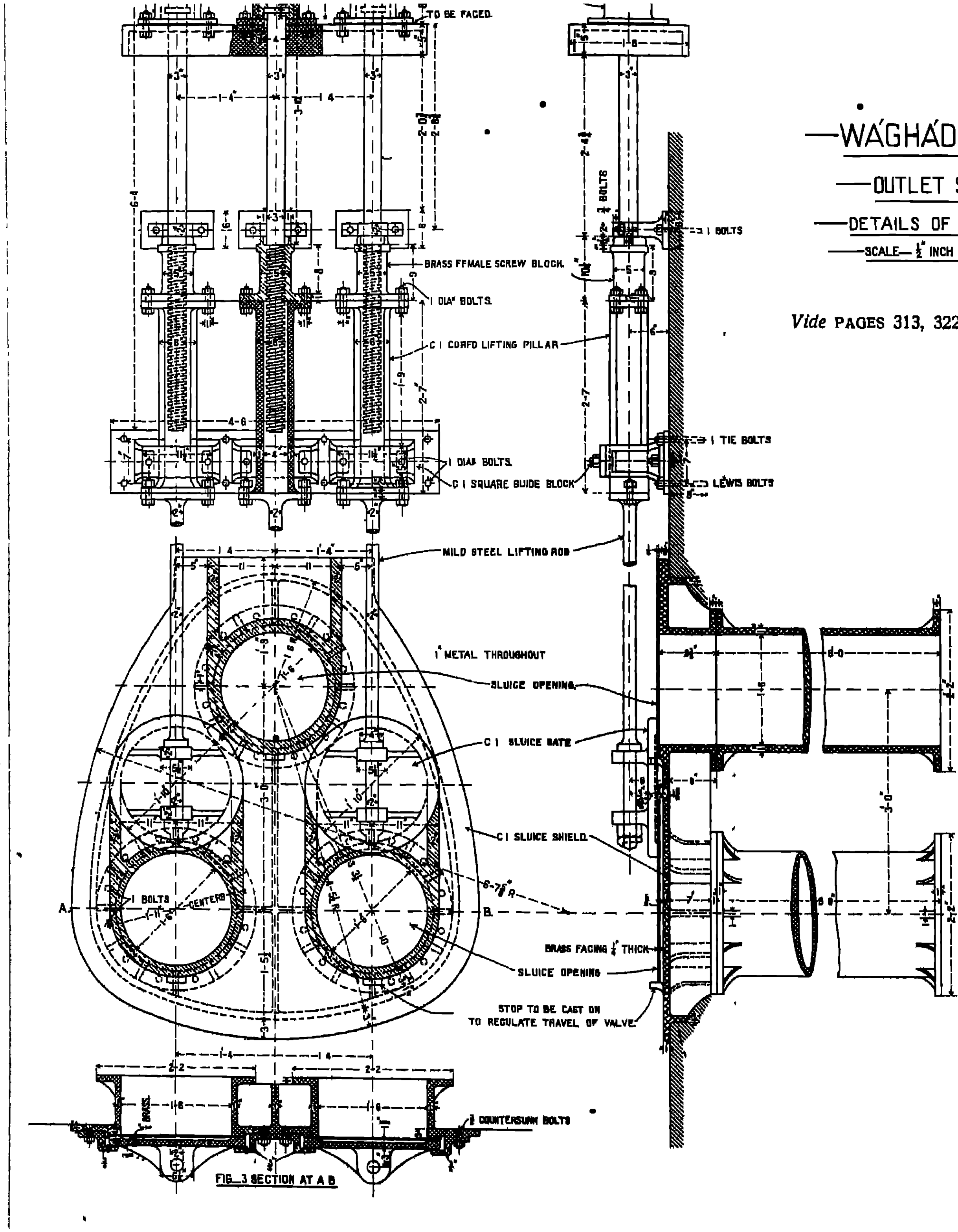
FIG_4 PLAN



Vide PAGES 286, 305 AND 311.



SCALE — 3/16 — 1 FOOT
FIG 3
DETAIL OF WROT IRON HANDLES
SET SCREW
FIG 4
FIG 5
OIL HOLE FOR COLLAR
1 EYE BOLT IN FLANGE THIS WITH A SMALL PIECE OF CHAIN FOR LOCKING HANDLE
FIG 13
GENERAL ELEVATION OF SLUICE GATE, ROD &c.
SCALE — 1/8 — 1 FOOT
FIG 14
DETAIL OF TOP OF STANDARD
FIG 6
ON AT G G
FIG 7
ON AT E E
FIG 8
DETAIL OF STEEL WEDGES
FIG 9
SECTION AT D D.
1 TURNED BOLT
STEEL WEDGES
SCREW 3/4 DIA AND 3/8 PITCH
STONE TO PROJECT 1½ BEYOND FACE OF WALL
FIG 16
DETAIL OF JOINT TO ROD
FIG 15
DETAIL OF BASE OF STANDARD
FIG 10 — DETAIL OF END OF LIFTING ROD.
ROUND STEEL ROD
FIG 17
SECTION THRO JOINT
TURNED BOLTS
FIG 20
DETAIL SHEWING FASTENING FACING TO CAST IRON
LEVEL OF SILL
SCALE — 3/4 — 1 FOOT
FIG 11
PLAN AT A A
FIG. 12
PLAN AT H. H
CHIPPING STRIP ABOUT 2" DEEP TO BE CAST AT TOP OF HOLE.
SURFACES LINED THICK ARE TO BE MACHINED
SCALE — 3/4 — 1 FOOT
FIG 18
DETAIL OF GUIDE TO ROUND ROD
FIG 19
DETAIL OF GUIDE TO SQUARE ROD.



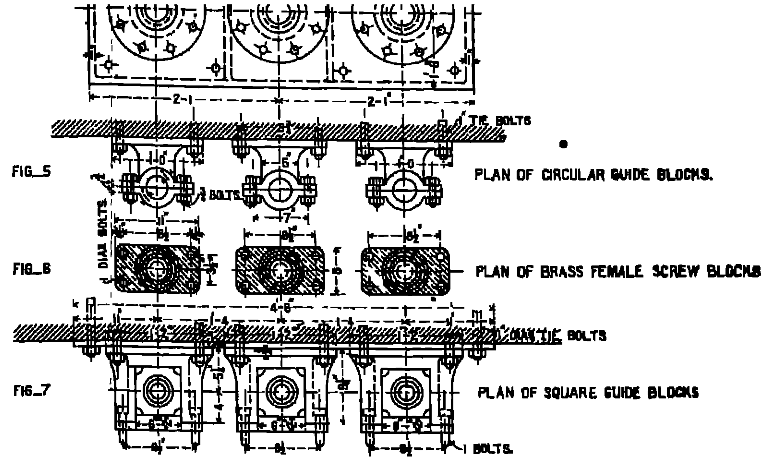
WAGHAD TANK

OUTLET SLUICE

DETAILS OF IRONWORK

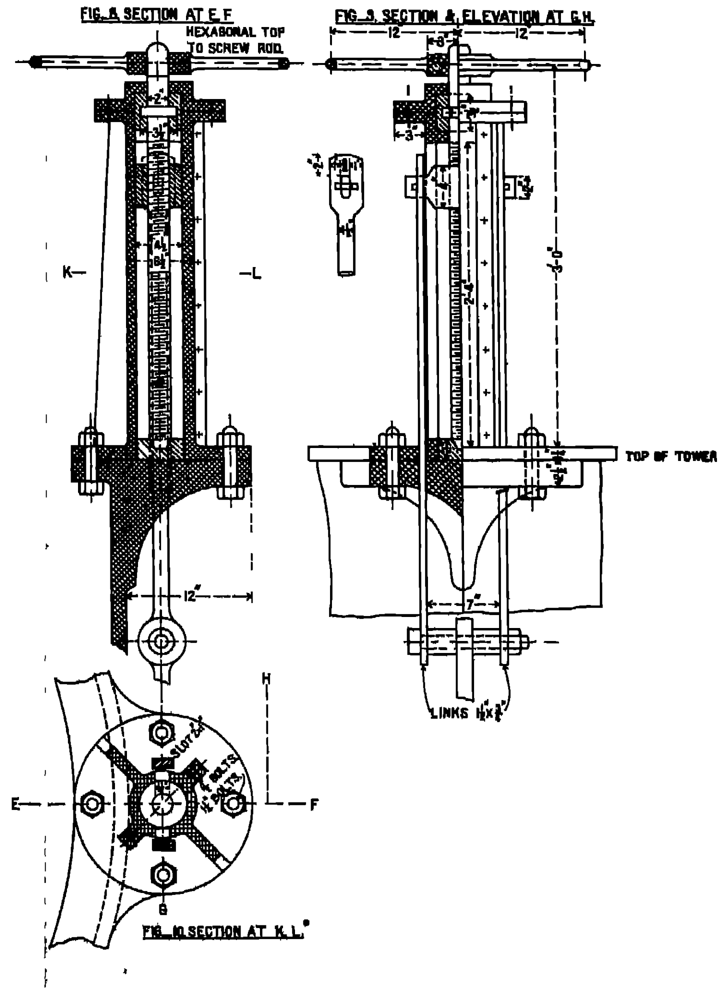
SCALE ½" INCH TO 1 FOOT

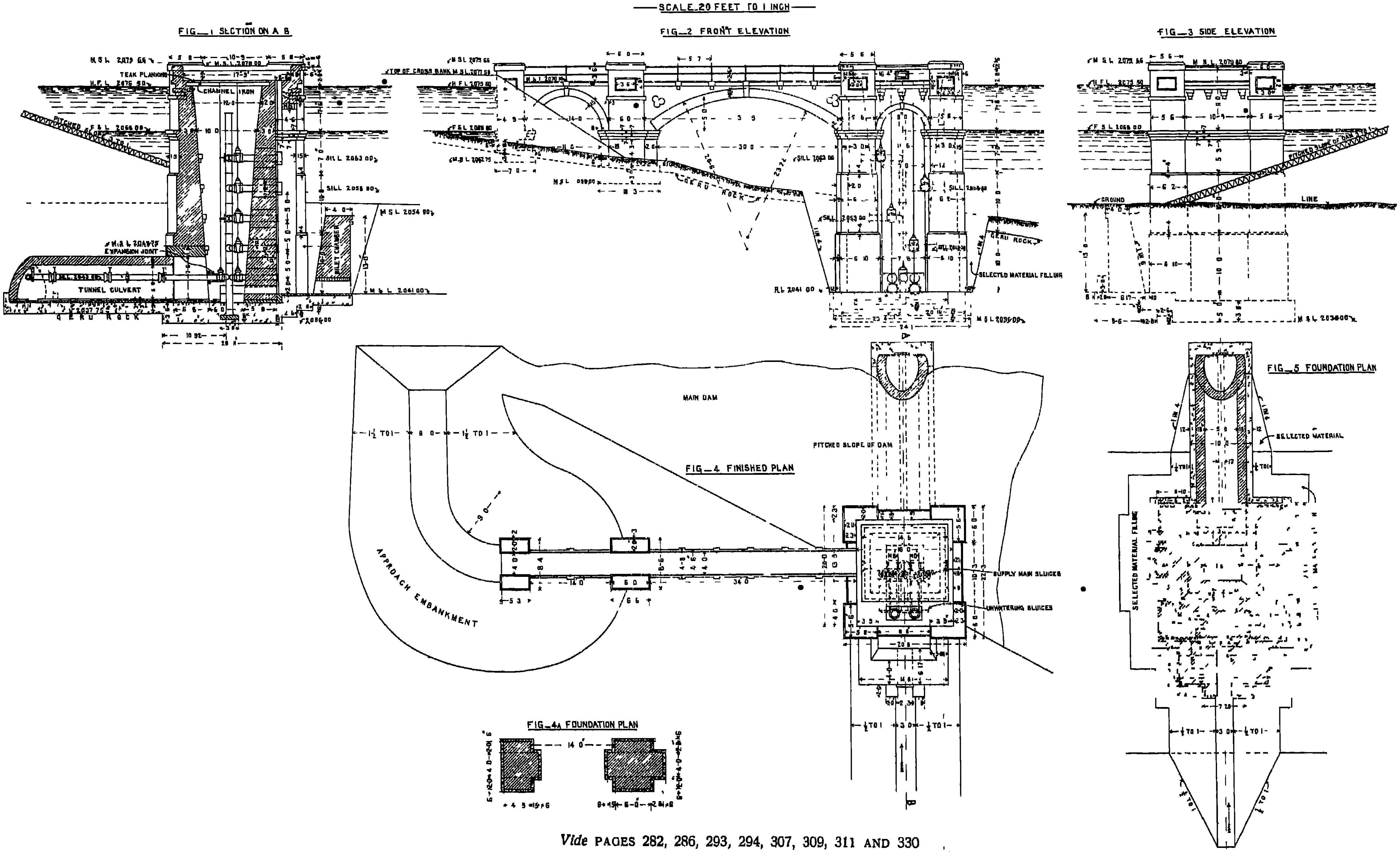
Vide PAGES 313, 322, 323 AND 324



BATEMAN'S CAPSTAN HEAD

SCALE ⅛ INCH TO 1 FOOT





Vide PAGES 282, 286, 293, 294, 307, 309, 311 AND 330

—OUTLET—

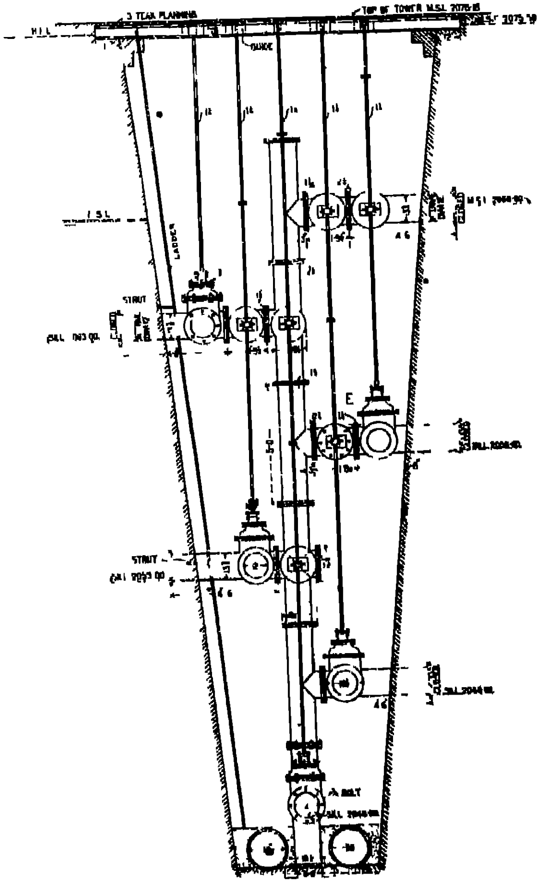
—DETAILED DRAWINGS OF IRONWORK—

—SCALE ¼ INCH TO 1 FOOT—

Vide PAGES 286, 289, 293 AND 330

—CROSS SECTION OF TOWER ON A.B.—

—SHOWING ARRANGEMENT OF DRAW-OFF VALVES A.B.—



- FIG 4 PLAN OF TOWER -

- SHOWING ARRANG. OF DRAW-OFF VALVES &c. -

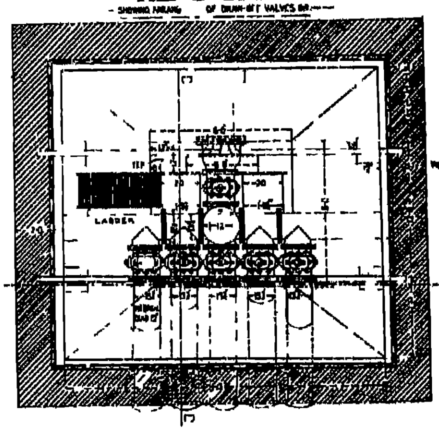


FIG 2

—DETAIL IN PLAN OF VERTICAL PIPE,—

DRAW OFF VALVES AND GUIDE AT E ON c.s A.B.

—SCALE ½ INCH TO 1 FOOT—

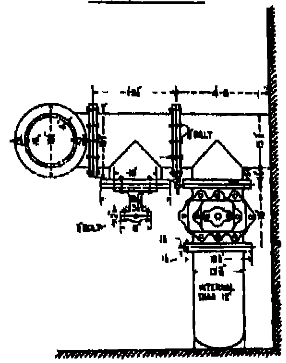


FIG -3

—DETAIL IN ELEVATION OF VERTICAL PIPE DRAW OFF VALVE GUIDE—

—ROD AND SUPPLY MAIN EXPANSION JOINT AT F ON c.s C.D—

—SCALE ½ INCH TO 1 FOOT—

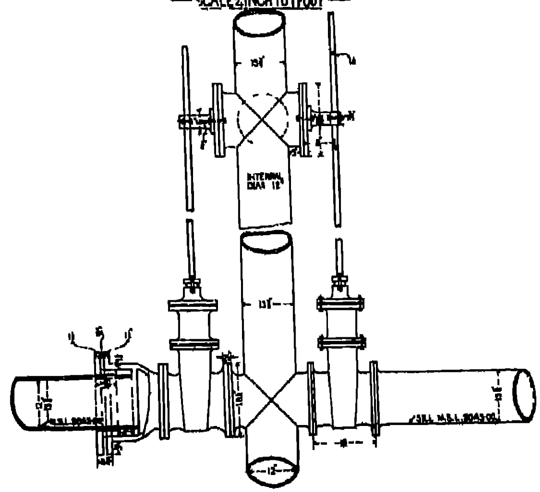
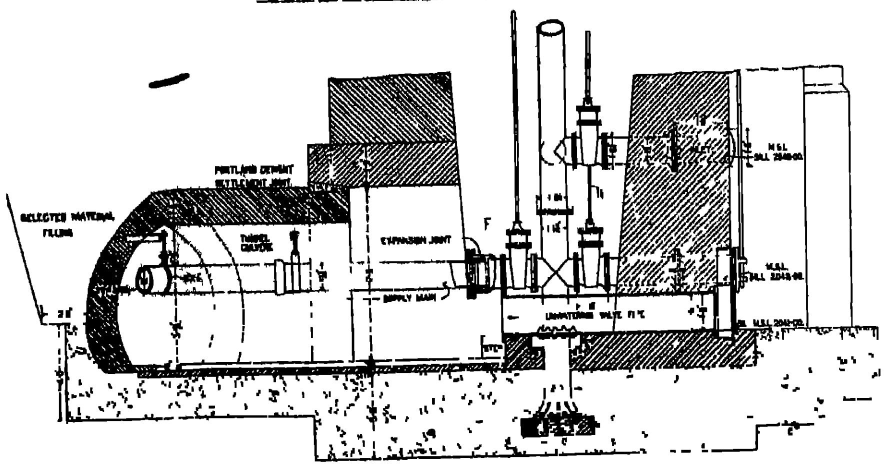


FIG -5

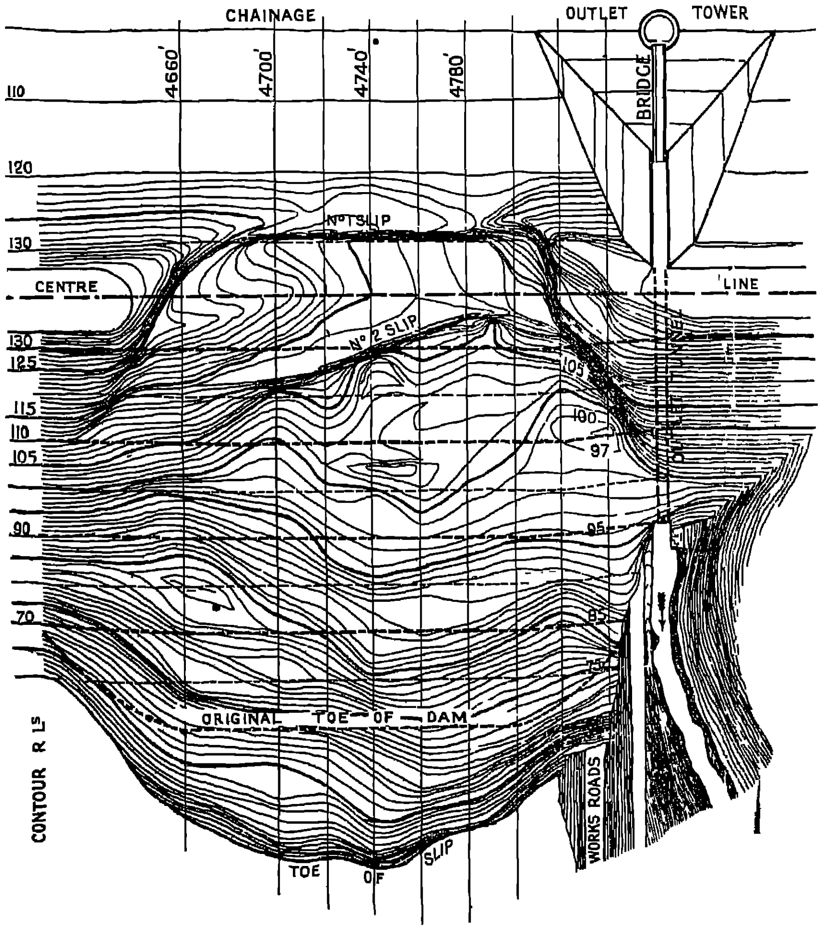
—CROSS SECTION OF BASE OF TOWER ON C.D.—

SHOWING ARRANGEMENT OF SUPPLY MAIN DRAW-OFF AND UNWATERING VALVES



SCALE 80 FT TO ONE INCH

CONTOURED PLAN OF DAM AFTER SLIP



NOTE
ORIGINAL CONTOURS DOTTED
SLIP CONTOURS IN FULL

Vide page 385

REARRANGED SLOPES OF DAM AFTER SLIP